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16. ABSTRACT

The Eleventh Annual Research Symposium on land disposal, remedial action, incineration and treatment of hazardous waste was held in Cincinnati, OH April 29 through May 1, 1985. The purpose of the Symposium was to present the latest significant research findings of ongoing and recently completed projects funded by the Land Pollution Control Division (LPCD), and the Alternative Technologies Division (ATD) of the Hazardous Waste Engineering Research Laboratory to persons concerned with hazardous waste management.

These proceedings are for Session A, Hazardous Waste Land Disposal and Session C, LPCD Posters. Papers presented by Symposium speakers and poster presentation Abstracts in the areas of (1) Remedial Action (2) Pollutant Assessment and (3) Pollutant Control are compiled. Subjects discussed include landfill design and operation, waste leaching and analyses, pollutant migration and control, waste modification, surface impoundments, flexible membrane liners, remedial action techniques and underground mine disposal.

This document covers Hazardous Waste Land Disposal only. A separate document concerned with Hazardous Waste Incineration and Treatment will be published by the Hazardous Waste Engineering Research Laboratory.

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FOREWORD

Today's rapidly developing and changing technologies and industrial products and practices frequently carry with them the increased generation of solid and hazardous wastes. These materials, if improperly dealt with, can threaten both public health and the environment. Abandoned waste sites and accidental releases of toxic and hazardous substances to the environment also have important environmental and public health implications. The Hazardous Waste Engineering Research Laboratory assists in providing an authoritative and defensible engineering basis for assessing and solving these problems. Its products support the policies, programs, and regulations of the Environmental Protection Agency, the permitting and other responsibilities of State and local governments, and the needs of both large and small businesses in handling their wastes responsibly and economically.

These Proceedings present the results of completed and ongoing research projects concerning the land disposal of hazardous waste.

David G. Stephan, Director
Hazardous Waste Engineering Research Laboratory

PREFACE

These Proceedings are intended to disseminate up-to-date information on extramural research projects concerning land disposal, remedial action and treatment of hazardous waste. These projects are funded by the U.S. Environmental Protection Agency's Office of Research and Development and have been reviewed in accordance with the requirements of EPA's Peer and Administrative Review Control System.

ABSTRACT

The Eleventh Annual Research Symposium on Land Disposal, Remedial Action, Incineration and Treatment of Hazardous Waste was held in Cincinnati, Ohio, April 29-May 1, 1985. The purpose of the symposium was to present to persons concerned with hazardous waste management the latest significant findings of ongoing and recently completed research projects funded by the Hazardous Waste Engineering Research Laboratory's Land Pollution Control Division and Alternative Technologies Division.

This volume is a compilation of speakers' papers and poster presenters' abstracts for Sessions A and C concerning hazardous waste land disposal. Areas covered are: 1) Remedial Action, 2) Pollutant Assessment, and 3) Pollution Control. Subjects include landfill design and operation, waste leaching and analyses, pollutant migration and control, waste modification, surface impoundments, technology assessments, remedial action techniques, and cost/economics.

This document covers hazardous waste land disposal only. A separate document for Session B, Hazardous Waste Incineration and Treatment, will be published by the Hazardous Waste Engineering Research Laboratory.

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TECHNICAL RESOURCE DOCUMENTS FOR HAZARDOUS WASTES

Norbert B. Schomaker
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Cincinnati, Ohio 45268

ABSTRACT

The Environmental Protection Agency is preparing documents to assist permit applicants and permit writers responsible for hazardous waste landfills, surface impoundments, land treatment facilities and waste piles. These Technical Resource Documents (TRDs) describe current technologies and methods for evaluating the performance of the applicant's design in the area of liners, leachate management systems, covers, and other aspects of the containment facilities. These documents are basically compilations of research efforts of the Land Pollution Control Division (LPCD) to date in these areas and are developed and published to assist in the implementation of the RCRA regulations concerning hazardous waste disposal facilities. The TRDs support RCRA Technical Guidance Documents prepared by EPA's Office of Solid Waste (OSW). The Guidance Documents provide details of design and operating procedures that contribute to the satisfactory performance of a facility.

INTRODUCTION

Land disposal of hazardous waste is subject to the requirements of subtitle C of the Resource Conservation and Recovery Act (RCRA) of 1976. This Act requires that the treatment, storage, or disposal of hazardous waste be carried out in accordance with RCRA regulations. Owners and operators of new facilities must apply for and receive a RCRA permit before beginning operation of such a facility. In reviewing and evaluating the permit application for acceptance or rejection, the permit official must make all decisions in a well defined and well documented manner. The TRD's are being developed and published to assist the permit applicants and permit review official to assure that the latest containment facility technology is being utilized. The TRD's are to be used in conjunction with the Technical Guidance Documents being prepared by OSW. These documents contain guidance, not regulations or requirements which the Agency believes comply with Design and Operating Requirements and the Closure and Post-Closure Requirements contained in Part 264 of the regulations. The information and guidance presented in these documents constitute a suggested approach for review and evaluation based on good engineering practices. There may be alternative and equivalent methods for conducting the review and evaluation. However, if the results of these methods differ from those of the Environmental Protection Agency's method, they may have to be validated by the applicant.

Eight TRDs have been completed to date. Six of the TRDs are related to landfills, and one document each is related to surface impoundments and land treatment. A listing of these documents is shown below, along with a brief description, publication number, and the contact person's name in parentheses.

TRD 1: EVALUATING COVER SYSTEMS FOR SOLID AND HAZARDOUS WASTE (SW-867) - This document presents a procedure for evaluating cover systems for solid and hazardous waste. It contains a checklist, with supporting documentation,

that a permit writer can use to evaluate proposed cover systems.
*055-000-00228-2 (R. E. Landreth)

TRD 2: HYDROLOGIC SIMULATION ON SOLID WASTE DISPOSAL SITES (SW-868) - This document provides a computer package to aid planners and designers by simulating hydrologic characteristics of landfill covers to predict percolation as a function of cover design and climate. This document has been updated to include the two-dimensional aspects of landfill cover systems. It is now identified as the "Hydrologic Evaluation of Landfill Performance (HELP) Model" SW-84-009 and SW-84-010. These documents have been published for public comment. The HELP model document will replace the SW-868 document. *055-000-00225-8 (D. C. Ammon)

TRD 3: LANDFILL AND SURFACE IMPOUNDMENT PERFORMANCE EVALUATION (SW-869) - The evaluation of leachate collection systems using compacted clay or synthetic liners to determine how much leachate will be collected and how much will seep through the liner into underlying soils is presented. The adequacy of sand and gravel drain layers, slope, and pipe spacing is also covered.
*055-000-00233-9 (M. H. Roulier)

TRD 4: LINING OF WASTE IMPOUNDMENT AND DISPOSAL FACILITIES (SW-870) - Information and guidance on liner systems is provided. The document discusses waste, liner types, compatibility, liner selection, specifications, design of leachate collection systems, and case study analysis methodology. It also includes a glossary of liner system related terms.
*055-000-00231-2 (R. E. Landreth)

TRD 5: MANAGEMENT OF HAZARDOUS WASTE LEACHATE (SW-871) - This document discusses leachate composition, leachate generation, selected management options, available treatment technologies, unit-treatment options, and system economics. It presents management options that a permit writer or hazardous waste landfill operator may consider in controlling and treating leachate.
*055-000-00224-0 (S. C. James)

TRD 6: GUIDE TO THE DISPOSAL OF CHEMICALLY STABILIZED & SOLIDIFIED WASTES (SW-872) - Basic information on stabilization/solidification of industrial wastes to insure safe burial of wastes containing harmful materials is presented. A summary of major physical and chemical properties of treated wastes is presented along with a listing of major suppliers of stabilization/solidification technology and a summary of each process.
*055-000-00226-6 (R. E. Landreth)

TRD 7: CLOSURE OF HAZARDOUS WASTE SURFACE IMPOUNDMENTS (SW-873) - The methods, tests, and procedures involved in closing a surface impoundment are discussed and referenced. Problems related to abandoned impoundments causing environmental degradation are discussed. Closure methods such as waste removal, consolidating the waste on site and securing the site as a landfill are also discussed.
*055-000-00227-4 (M. H. Roulier)

TRD 8: HAZARDOUS WASTE LAND TREATMENT (SW-874) - Land treatment medium, characteristics of hazardous waste, allowable waste loadings, facility planning for land treatment, land treatment permit processes, closure and items to be considered during permit evaluation are discussed.
*055-000-00232-1 (C. C. Wiles)

Each of the completed TRDs is described in greater detail in the following section of this paper.

TECHNICAL RESOURCE DOCUMENT SUMMARIES

Evaluating Cover Systems for Solid and Hazardous Waste (SW-867)

A critical part of the sequence of designing, constructing, and maintaining an effective cover over solid and hazardous waste is the evaluation of engineering plans. This TRD presents a procedure for evaluating closure covers on solid and hazardous wastes. All

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*These documents have been published and the reports are available from GPO by requesting the stock number. Copies can be obtained for a price from the:

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aspects of covers are addressed in detail to allow for a complete evaluation of the entire cover system.

Covers may be evaluated for conformance to requirements for closure covers by the following eleven (11) general procedures in sequence:

1) examine soil test data,
2) examine topography,
3) examine climate data,
4) evaluate composition,
5) evaluate thickness,
6) evaluate placement,
7) evaluate configuration,
8) evaluate drainage,
9) evaluate vegetation,
10) evaluate post-closure plan, and
11) evaluate contingencies plan.

This document describes current technology for landfill covers in three broad areas: Data Examination, Evaluation Steps and Post-Closure Plan. The data examination discusses test data review procedure, topographical data review and climatological data review procedures. The evaluation steps include cover composition, thickness, placement, configuration, drainage and vegetation. The post-closure aspects include maintenance and contingency plan evaluation procedure. There are 36 specific steps, regarding the preceding factors, which are recommended to be followed in evaluating a permit for a cover for hazardous waste.

Hydrologic Simulation on Solid Waste Disposal Sites (SW-868)

Percolation and runoff of precipitation at solid waste disposal sites create a potential for ground and surface water contamination with leachate. The proper landfill site design and operational approach is to minimize or eliminate percolation through the solid waste and thus control the formation of leachate.

The purpose of this TRD is to provide an interactive computer program for simulating the hydrologic characteristics of a solid and hazardous waste disposal site operation. A large number of cities within the U.S. for which five years of climatic records exist have been put on tape for easy access and can be used in lieu of on-site measurements. In addition, the model stores many default values of parameter estimates which can be used when measurements and existing data files are not available. The user must supply the geographic location, site area and hydrologic length, the characteristics of the final soil and vegetative cover, and default overrides where deemed necessary. From minimal input data, the model can simulate daily, monthly, and annual runoff, deep percolation, temperature, soil water, and evapotranspiration.

The model, which is a modification of the Soil Conservation Service (SCS) number method for predicting runoff and the hydrologic portion of the U.S. Department of Agriculture, Science and Education Administration (USDA-SEA) hydrologic model entitled "Chemicals, Runoff, and Erosion from Agricultural Management Systems (CREAMS)," has been modified to conform to the design characteristics of solid and hazardous waste disposal sites. The model takes engineering, hydrologic, and climatologic input data in the form of rainfall, average temperature, solar radiation, and leaf area indices and characteristics of cover material and performs a sequential analysis to derive a water budget including runoff, percolation, and evapotranspiration.

The user can specify up to three soil layers and may also specify a membrane liner at the base of the cover. The decreasing effectiveness of the liner is simulated. The model is designed for use in a conversational manner, that is, the user interacts directly with the program and receives output immediately. No prior experience with computer programming is required. All commands necessary to use the model are presented in the user's manual. The model can also be run in batch mode, which requires some computer programming experience.

Landfill and Surface Impoundment Performance Evaluation (SW-869)

This TRD describes how to evaluate the capability of various liner and drain designs to control leachate release. The author has allowed for the widely varied technical backgrounds of his intended audience by presenting, in full, the rigorous mathematics involved in reaching his final equations. Thus, any evaluator can take full advantage of the manual up to the level of his own mathematical proficiency.

This evaluation system considers the transport of liquids through simple modular configurations. The important parts of these modules are identified as the Liquid Transport Control (LTC) system, and the Diversion Interface (DI). The mathematics consider the efficiency with which the LTC and DI components convert leachate flow from vertical to horizontal. The extent of ponding on a bottom liner is considered in relation to the position and effectiveness of the drainage system.

The procedures described in the document should allow an evaluator to determine the adequacy of design for:

1) compacted clay liners or synthetic liners intended to impede the vertical flow of fluids,
2) sand or gravel drainage layers intended to convey liquids laterally into collection systems,
3) slopes on such liner systems, and
4) spacings of collection drains.

The evaluation procedure is independent of the structure of the regulation it supports. If regulations specify acceptable rates of release for leachate, or efficiencies of leachate collection systems, calculation results may be used directly to determine if the landfill is acceptable. However, if the regulations describe acceptable environmental impact, results from the evaluation procedure can be used as input for further evaluation in other manuals to determine if the impact of the landfill is acceptable.

Lining of Waste Impoundment and Disposal Facilities (SW-870)

This document provides information on performance, selection, and installation of specific liners and cover materials for specific disposal situations, based upon the current state-of-the-art of liner technology and other pertinent technologies. It contains descriptions of wastes and their effects on linings, a full description of various natural and artificial liners; service life and failure mechanisms; installation problems and requirements of liner types; costs of liners and installation; and tests that are necessary for pre-installation and monitoring surveys.

Described in the report are potentially useful lining materials and their associated technologies, including remolded and compacted soils and clays, admixes, polymeric membrane liners, sprayed-on liners, soil sealants, and chemisorptive liners. The specific characteristics of these liners in service environments with various types of wastes are discussed, particularly with respect to compatibility with wastes, permeability to water and waste, failure mechanisms, and service life.

Design and construction of waste facilities with various types of linings are described, emphasizing installation of membrane liners and their problems. Special problems associated with the design and construction of solid waste disposal sites are considered. The subjects of leachate collection above the liner, gas control, and top-seals are addressed.

Several processes for selecting a specific liner or various satisfactory liners for a given disposal site are proposed. These include the use of compatibility tests, moderate duration exposure tests, soil conditions tests, costs, etc. Methods for the preparation of specifications covering the selected liners or groups of liners are suggested.

In the appendices data are presented on a soil classification system; liner industry organization; selected liner test methods; liner installation; leachate collection; soil liner design; case study analysis and specification for liner membranes. As new technology is developed, and as experience in the use of liners becomes available, the manual will be upgraded on a chapter-by-chapter basis.

Management of Hazardous Waste Leachate (SW-871)

This document has been prepared to

provide guidance for permit officials and disposal site operators on available management options for controlling, treating, and disposing of hazardous waste leachates. It discusses considerations necessary to develop sound management plans for leachate generated at surface impoundments and landfills. Management may take the form of leachate collection and treatment, or pre-treatment of the wastes.

The manual addresses areas that must be considered in evaluating a leachate management option: leachate generation and composition, and available technologies for treatment of leachate and hazardous waste.

Leachate composition is examined for several hazardous waste landfills, synthetic leachates from lab studies, and sanitary landfill leachates. The expected composition of the leachate and the concentrations of the pollutants in the leachate are presented along with possible guidelines for permit writers to use in determining the relative hazards of a particular leachate.

A key section enumerates factors which influence treatment process selections and provides a suggested approach for systematically addressing each. Selected hypothetical and actual leachate situations are used as examples for applying the approach to the selection of appropriate treatment processes.

Other sections address monitoring, safety, contingency plans/emergency provisions, equipment redundancy/backup, permits, and surface runoff.

Guide to the Disposal of Chemically Stabilized and Solidified Waste (SW-872)

Stabilization/solidification of industrial waste is a pretreatment process designed to insure safe disposal of wastes containing harmful materials. The purpose of this TRD is to provide guidance in the use of chemical stabilization/solidification techniques for limiting hazards posed by toxic wastes in the environment, and to assist in the evaluation of permit applications related to this disposal technology. The document addresses the treatment of hazardous waste for disposal or long term storage and surveys the current state and effectiveness of waste treatment technology. A summary of the major physical and chemical properties of treated wastes is presented. A listing of major suppliers of stabilization/solidification technology and a summary of each process is included.

This guide provides the background information needed for waste generators and regulatory officials to determine the testing program and/or product information necessary for them to make the best engineering judgments concerning the long term effectiveness in site-specific conditions. The user should have some familiarity with general soil characteristics, water balance, climatic conditions, and fundamentals of leachate generation.

Closure of Hazardous Waste Surface Impoundments (SW-873)

"Closure of Hazardous Waste Surface Impoundments" presents a listing of closure plan and post-closure care considerations and details for surface impoundments containing hazardous wastes. It is written primarily for staff members in EPA Regional offices or state regulatory offices, who are charged with evaluating and approving closure plans for surface impoundments under regulations of the Resource Conservation and Recovery Act of 1976. Methods of assessing site closure considerations are documented.

The guide describes and references the methods, tests, and procedures involved in closing a site in such a manner that (a) minimizes the need for further maintenance, and (b) controls, minimizes, or eliminates, to the extent necessary to protect human health and the environment, post-closure escape of hazardous waste, waste constituents, leachate, contaminated rainfall or waste decomposition products to groundwater, surface water, or the atmosphere. Problems that have been overlooked in abandoned impoundments and have caused environmental damage are discussed. The techniques involved are pertinent to closing an impoundment either by removing the hazardous waste or by consolidating the waste on-site and securing the site as a landfill. Technical criteria for implementing the closure, specifically those regarding aspects substantially different from a landfill, are given. Relevant literature or procedures are documented for more in depth review.

Hazardous Waste Land Treatment (SW-874)

The objectives of "Hazardous Waste Land Treatment" are to describe current technology and to provide methods for evaluating the performance of an applicant's hazardous waste land treatment facility design. Land treatment is approached comprehensively from initial site selection through final closure, and additional information sources are referenced liberally. Land treatment, which involves using the surface soil as the treatment medium, is already widely practiced by some industries for handling their hazardous wastes. However, the lack of systematic study or monitoring of many past or present facilities has limited the knowledge and data base with regard to the important parameters and interactions. Additionally, many potentially land treatable wastes have not been tested or have been examined only under a limited range of conditions. Therefore, information is needed on site selection, waste characteristics, optimum soil and climatic conditions for degradation of organic constituents, application rates and scheduling, decomposition products, and the contingency for environmental contamination. Potential problems include worker's health and safety, organic components, water pollution hazards, and local atmospheric emissions.

Land treatment of hazardous waste has not been under close scrutiny for a long enough period of time to acquire numerous documented failures resulting in environmental pollution. The potential for such failures due to the lack of proper design and/or management is evident from research conducted on the land treatment of other nonhazardous waste materials. For land treatment to be an effective system, the process must operate within a narrow range of parameters. Exceeding them could result in the uncontrolled release of pollutants to the environment.

FUTURE TECHNICAL RESOURCE DOCUMENTS

Additional TRDs now being developed or in planning stages by the Office of Research and Development:

A listing of these documents is shown below, along with a brief description and the project officer's name in parentheses.

1. SOIL PROPERTIES, CLASSIFICATION AND HYDRAULIC CONDUCTIVITY TESTING

This report is a compilation of available laboratory and field testing methods for the measurement of hydraulic conductivity (permeability) of soils. Background information on soil classification, soil water, and soil compaction are included along with descriptions of sixteen methods for determination of saturated or unsaturated hydraulic conductivity. This TRD (SW-925) was published by OSW in March 1984 for public comment. It is being revised to incorporate public comments that were received. The revised version will be published through the Government Printing Office in late 1985. (M.H. Roulier)

2. SOLID WASTE LEACHING PROCEDURES MANUAL

This is a report on laboratory batch procedures for extracting or leaching a sample of solid waste so that the composition of the lab leachate is similar to the composition of leachate from waste under field conditions. This TRD (SW-924) was published by OSW in March 1984 for public comment. It is being revised to incorporate public comments that were received. The revised version will be available through the Government Printing Office in late 1985. (M. H. Roulier)

3. BATCH SOIL PROCEDURE TO DESIGN CLAY LINERS FOR POLLUTANT REMOVAL

This project is studying the feasibility of using soil/leachate batch adsorption procedures for designing compacted clay liners to remove/retain known amounts of pollutants, and will contribute to information on designing and evaluating clay liners to limit pollutant release from landfills. The project will be completed in September 1985. (M. H. Roulier)

4. METHODS FOR THE PREDICTION OF LEACHATE PLUME MIGRATION AND MIXING

This project has developed a variety of computer programs for hand-held calculators, microcomputers, and macrocomputers. The programs predict leachate plume migration from single and multiple sources. The document also contains discussions of sorption, case histories and a field study. A draft for public comment will be completed in late 1985. (M. H. Roulier)

5. HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL

The HELP Model is a modification of the original waste disposal site hydrologic model entitled,"Hydrologic Simulation on Solid Waste Disposal Sites." This update has incorporated the two-dimensional aspects of landfill cover systems, as well as the addition of the leachate collection system. OSW published this TRD (SW-84-009 and SW-84-010) for public comment in two volumes. The two volumes include the user's guide for version 1 and documentation and description of the program. Version 2 of the HELP Model is being developed to incorporate public comments and results from verification studies and will be published in early 1986. (D.C. Ammon)

6. DESIGN, CONSTRUCTION, MAINTENANCE, AND EVALUATION OF CLAY LINERS FOR HAZARDOUS WASTE FACILITIES

This project has gathered and assembled information on construction and use of compacted soil liners, including test methods, chemical compatibility, failure mechanisms, performance of existing facilities, and field quality assurance and inspection to insure that the liner performs as designed. The project is scheduled for completion in May 1985. (M. H. Roulier)

CONCLUSION

The Technical Resource Documents (TRDs) that are being prepared by the Land Pollution Control Division (LPCD) are a series of documents which provide design, operation, and evaluation information related to hazardous waste disposal facilities to assist the regulated community and the permitting authorities. Eight documents have been completed with six more in process by the Office of Research and Development. These documents describe state-of-the-art summaries of technologies and evaluation techniques determined by the Agency to constitute good engineering design, practices and procedures. The TRDs present the sum total of the body of information and experience gained by the Agency over the years on a given topic. As such, they contain factual summaries concerning the experience and effectiveness of design alternatives, covering what has been found not to work as well as what has been found to be effective. The TRDs can be considered as technical background or development documents supporting the RCRA guidance documents and regulations As new information is developed, the Agency intends to update each of these documents so that they reflect the latest state-of-the-art information.

The project officers can be contacted by writing or telephoning the: U. S. EPA, Hazardous Waste Engineering Research Laboratory, Land Pollution Control Division, 26 West St. Clair Street, Cincinnati, Ohio 45268, Telephone (513) 684-7871.

EVALUATION OF CHEMICAL GROUT INJECTION TECHNIQUES FOR HAZARDOUS WASTE CONTAINMENT

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ABSTRACT

Field tests were conducted with two chemical grouts to determine if a continuous impermeable seal could be developed by injecting grout into coarse-grained soils. Acrylate and sodium silicate grouts were injected into separate 2 x 4 m test beds of medium sand at a 30-50 cm depth using a maximum hole spacing of 40 cm (center-to-center) to produce a pan-like layer of grout. While some silicate grout bulbs coalesced, there were significant gaps in the grout layer and shrinkage of grout produced nonuniform cementing. The acrylate grout flowed to the bottom of the sand bed and did not produce grout bulbs.

In a large field test, 30-percent sodium silicate was injected at 2.44 m depth in fine sand using injection holes on 1.52 m-centers. Significant problems were noted when the pods were excavated. The pods were not symmetrical and ungrouted areas were present between pods. Pockets of coarse sand in the finer sand were not cemented although they had been filled with grout. Root and rootlet holes in the cemented sand were not sealed.

Conventional injection of chemical grouts did not produce an impervious seal. More unusual techniques, such as jet grouting, may be needed to obtain a useful seal.

INTRODUCTION

Chemical grouts have been widely employed in adding physical strength to foundation materials and reducing the permeability of soils(1,2). The use of chemical grouts in sealing soils around hazardous waste sites is an extension of existing techniques; but very specific problems that relate to grout solidification, durability, and techniques for grout injection must be treated when hazardous waste sites are being sealed. Previous reports from this work unit(3,4,5) have discussed the problems of grout setting and the physical durability of grout after placement.

PURPOSE

The present report discusses problems associated with the injection of selected grouts in sand test beds and a larger-scale field test where chemical grout (sodium silicate) was injected into an 85-sq m test area. In both operations the objective was to evaluate the ability of standard grout injection techniques to produce a continuous layer of grouted sand that could potentially be used as a barrier to waste movement. None of the test areas contained hazardous wastes.

APPROACH

The two chemical grouts (acrylate and sodium silicate) that showed the greatest potential for successful application based on their ability to set and to remain unaltered in the presence of typical wastes were selected for further field-scale injection. The extended testing was designed to determine if the chemical grouts could be made to form a continuous seal when injected in sand. Only conventional injection techniques using grout pumps and shallow injection points were employed.

Small-Scale Grouting Field Tests

Two types of grout, acrylate and sodium silicate, were selected for small-scale field testing to see if coalescing grout bulbs could be produced. These two grouts were considered as test candidates because they were relatively easy to control in terms of gelling times and showed some ability to set or gel in the presence of potential contaminants, although displacement of contaminants with clean water may be necessary to avoid waste/grout compatibility problems(4).

The small-scale tests were performed by injecting each grout into a medium, well-sorted sand in a separate bin that contained a 1-m deep layer of sand. Each test plot covered an area 2 x 4 m. Grout was injected at a depth of 30 to 50 cm in a grid-like pattern (see diagram in Reference 4) with a maximum spacing of 40 cm between hole centers (Figure 1). The objective of the test was to produce a pan-like continuous seal.

The silicate grout used was a 30 percent solution of JM-grade sodium silicate (technical). The hardener employed was a proprietary mixture containing calcium chloride, magnesium chloride, and dimethylformamide. The chemical grout was made up using tap water.

The acrylate grout was made up from a proprietary mixture of acrylate monomer and methylenebisacrylate. Ammonium persulfate was used as an initiator. The gel was also made up with tap water.

Both grouts were injected using a progressive cavity pump and each injection hole received approximately 7 liters of grout. Gel times were established in the laboratory to allow five minutes per batch for mixing and grout injection. Each 7-liter batch was mixed and injected separately so that gel time could be checked. In all cases the test gel time was identical to or less than the design gel time. No simulated wastes were present, and no interference from the test bed material was observed.

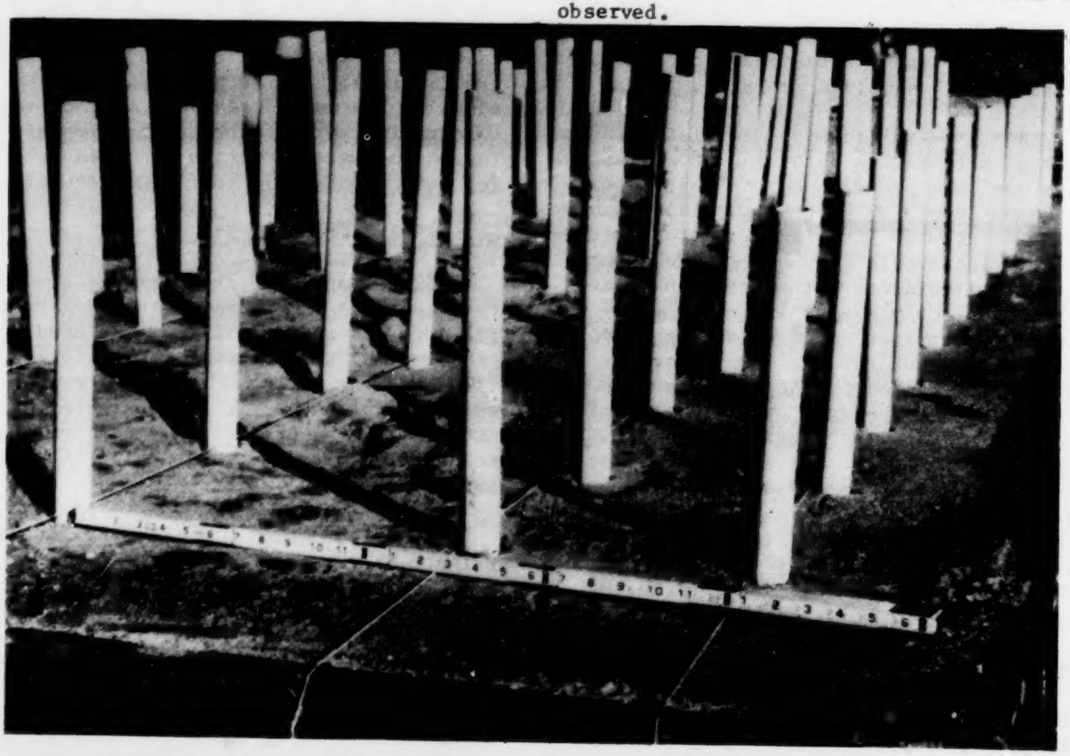

Figure 1. Photo of test bed for small-scale test sealing. The pipe sections mark the grout injection points.

The test beds were a well-sorted, medium-grained quartz sand with less than 10 percent pebble-sized material (Figure 2). The sand was wet to field capacity. The test beds were saturated but allowed to drain prior to grout injection. A 7-cm diameter slotted plastic pipe was implaced under each bed to allow free drainage. After grout injection the test beds were covered with a plastic tarp to allow the grout to gel and cure without being washed out by rainwater. During both grouting operations, water and grout were observed to be flowing out of the test bed drains suggesting displaced capillary water and some of the grout was being lost to the bottom drains. The grout injection pipe was withdrawn from the sand after grouting and in all cases some liquid grout remained in the injection holes indicating the bottom and side walls of the hole had been saturated with the grout. In both of the freshly grouted test beds there was evidence of gel in the surface sand around the hole; but only the sodium silicate made a cohesive pod of material around the grout hole. In the acrylate material thin rubbery strands of gel could be observed when the surface sand was examined, but the injected sand had little cohesion.

Both the silicate and acrylate grouted beds were tested to determine if a continuous seal had been formed. A shallow (10-cm deep) trench was dug on the loose sand over the grouted layer and water containing a dye (fluorescein) was added. To assure that the liquid did not overtop the pan-like layer, only 80-100 liters of water was placed in the trench and water was added only when all of the dye had drained through the bottom of the trench and no drainage water appeared at the bottom of the test unit. The times required for the appearance of dye at the test bed drain and the quantities of water and dye added were not significantly different for the grouted beds when compared to a similar but ungrouted sand bed. The dye testing indicated that no continuous seal had formed.

The grouted test beds were excavated and the position and character of the grouted material was noted. In the sodium silicate test bed two problems were apparent; gaps existed between adjacent grout bulbs where water could migrate around grout bulbs into the drainage layer (Figure 3) and the center portions of many of the grout bulbs were not cemented (Figure 4). The grout bulbs in many cases were in contact, but enough gaps existed to allow the rapid movement of dye through the grouted layer. The lack of cementation in the central portion of the pods is more difficult to explain. The phenomenon is probably caused by shrinkage due to water loss (syneresis), but the gel is concentrating on the outside of the sand mass and did not shrink to the inside of the mass.

The acrylate grout did not form pods and very little grout appeared in the upper sand layer. The major part of the acrylate grout was found in irregular masses at the bottom of the test bed (Figure 5). The grout had not plugged the slotted pipe drain but was covering portions of the pipe. The dye could easily pass through the injected layer into the drain. The grout had not created a barrier to dye movement. Test injections of the acrylate grout indicated that gelling occurred within minutes of injection and inspection of the sand on freshly grouted holes indicated gel was present in sand at the surface in the immediate area of the hole at the time of injection. The gel did not appear to adhere to the sand and may have migrated to the lower part of the test bed. The water added to the cart during dye testing may account for some of the grout movement, although the water addition occurred after gel formation should have been complete.

Large-Scale Grouting Field Tests

Experience with the small-scale sand test beds was incorporated into a larger field test. Sodium silicate was used for grouting and a field test site underlain by fine sand was selected to minimize problems of grout shrinkage(6). To maximize the probability for producing coalescing grout pods, all holes were grouted to refusal or until grout take was 0.76 cu m.

An 85-sq m plot underlain by fine sand (Figure 6) was made available for this study on a test area at Fork Polk, LA. Ten holes were spaced on 1.52-m centers using the pattern shown in Figure 7. Injection holes were drilled with a 10.2-cm-diameter auger to a depth of 2.75 m and the hole was backfilled with coarse filter sand to a depth of 2.44 m. A 3.05-m section of 5-cm plastic pipe was inserted in the boring and the outside of the borehole was sealed with pelleted and powdered bentonite. Water was added after each batch of bentonite to

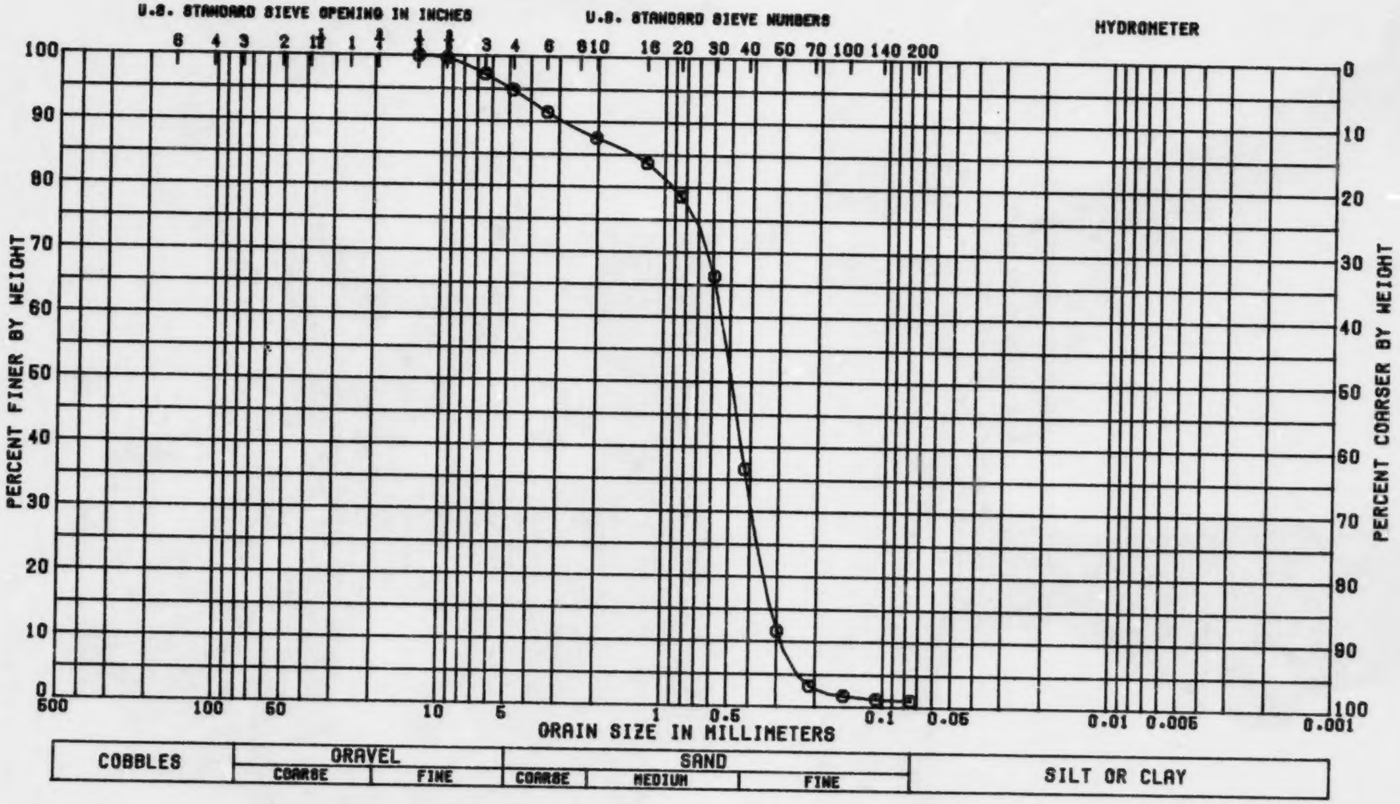

Figure 2. Grain-size distribution of sand used for the small-scale field test.

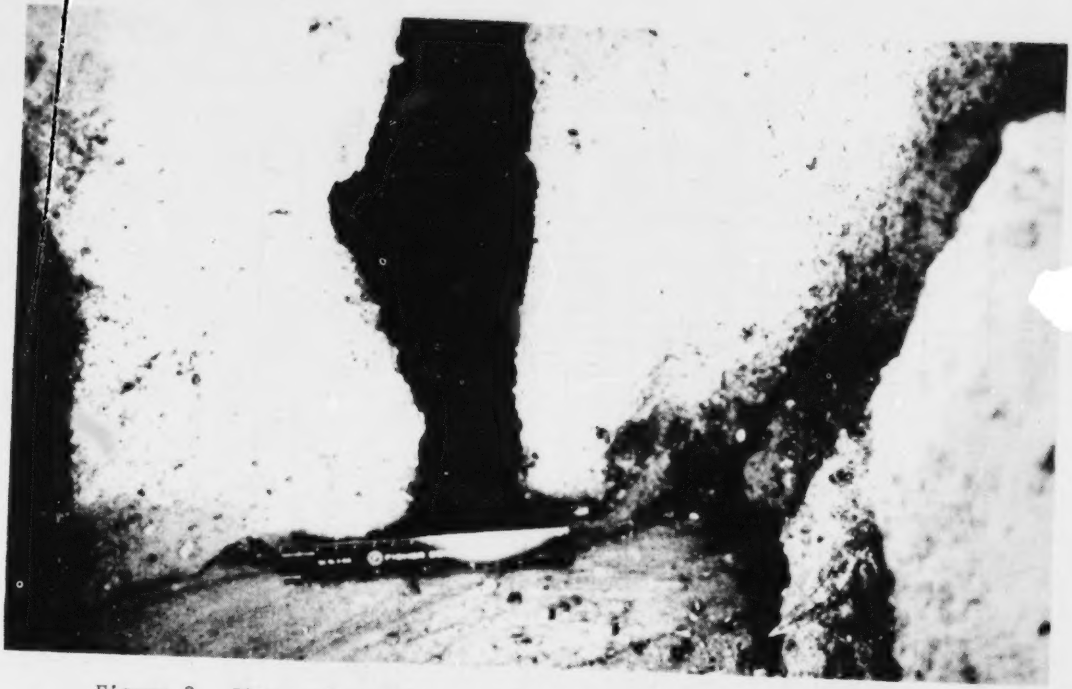

Figure 3. Photo of gaps between adjacent grout bulbs observed in the small-scale field test.

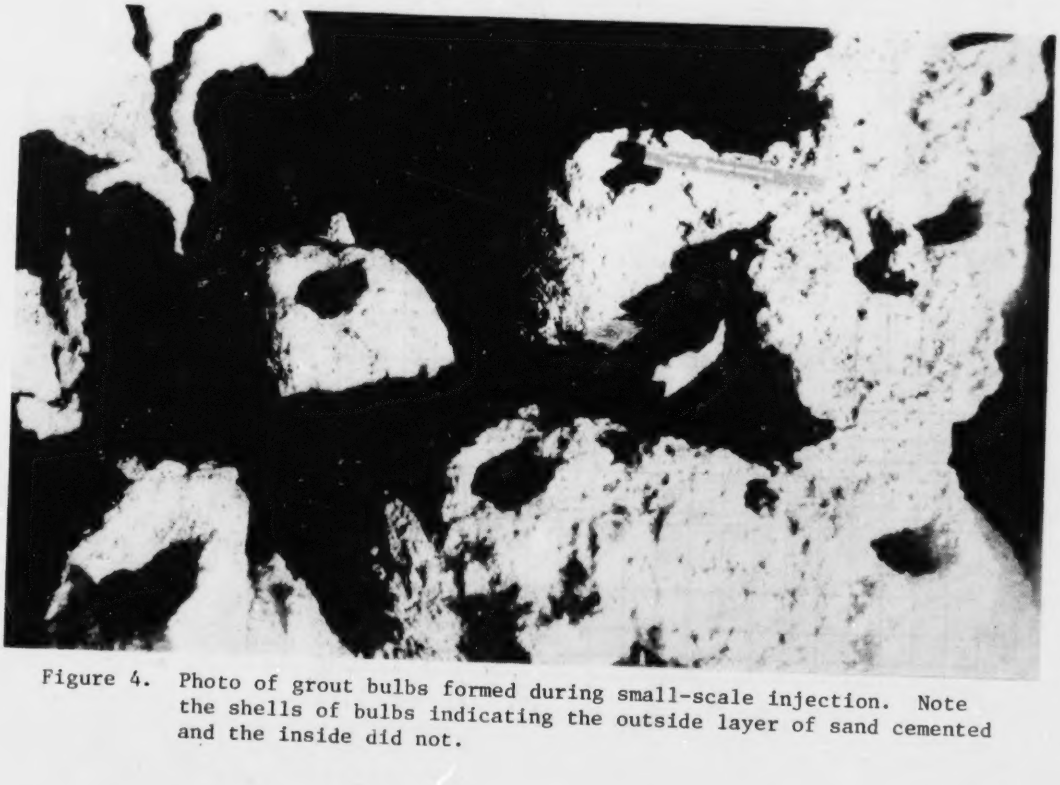

Figure 4. Photo of grout bulbs formed during small-scale injection. Note the shells of bulbs indicating the outside layer of sand cemented and the inside did not.

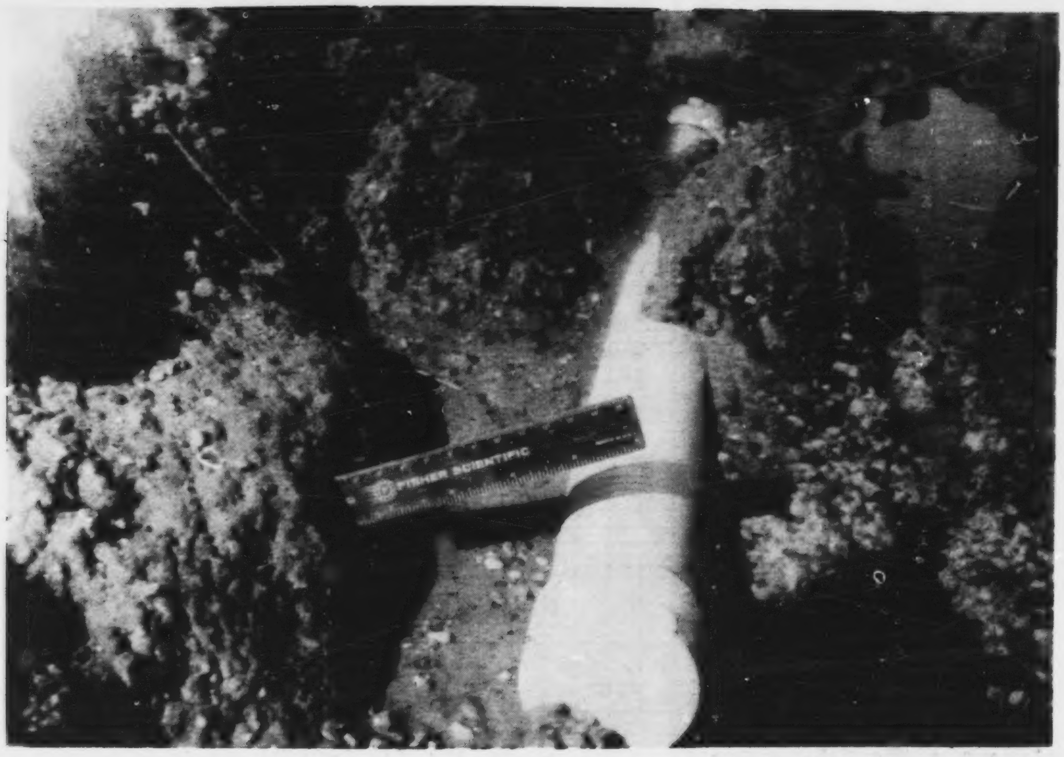
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Figure 5. Photo of the sand mass at the bottom of the test bed cemented with acrylate gel.

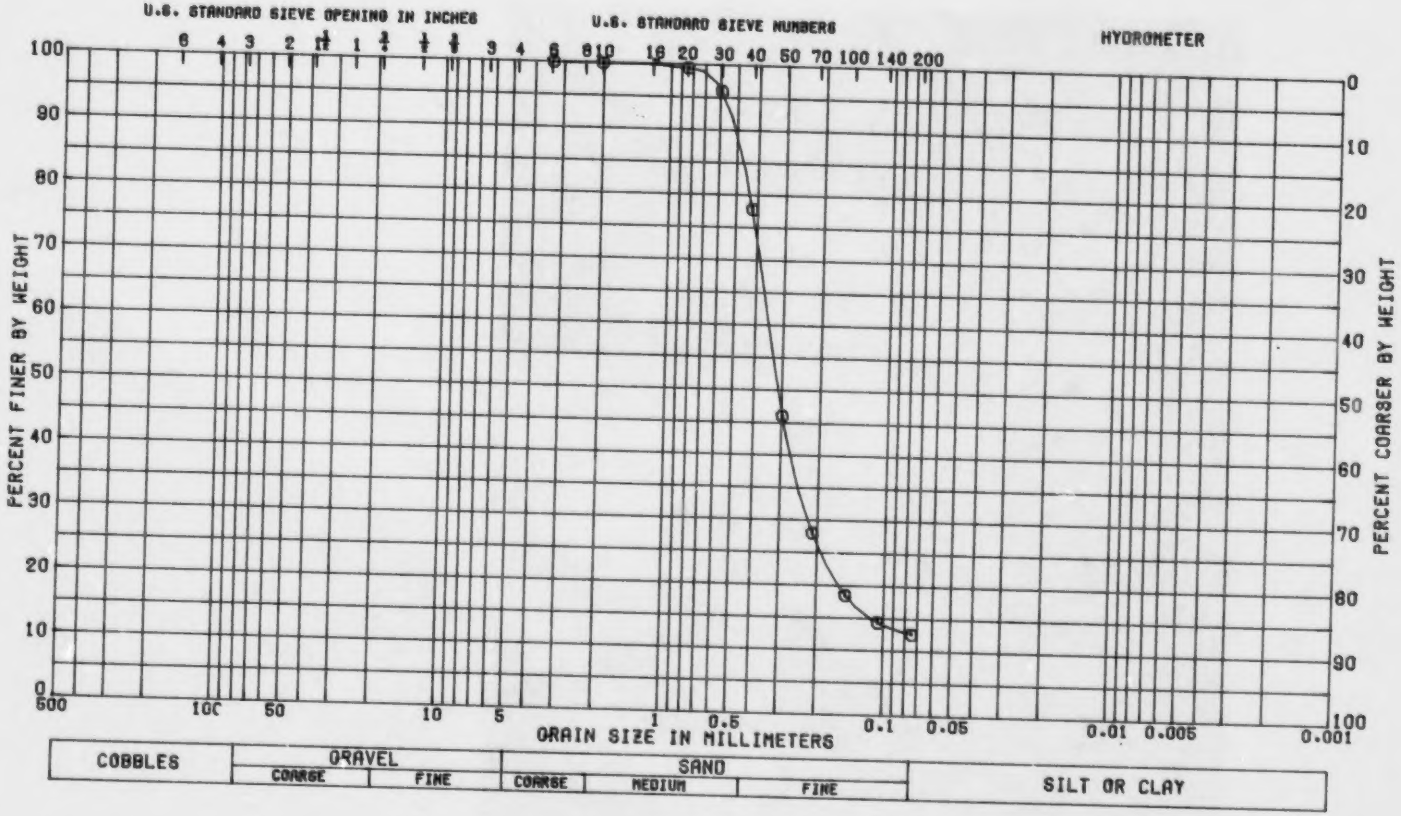

Figure 6. Grain-size distribution of sand under the larger-scale test area.

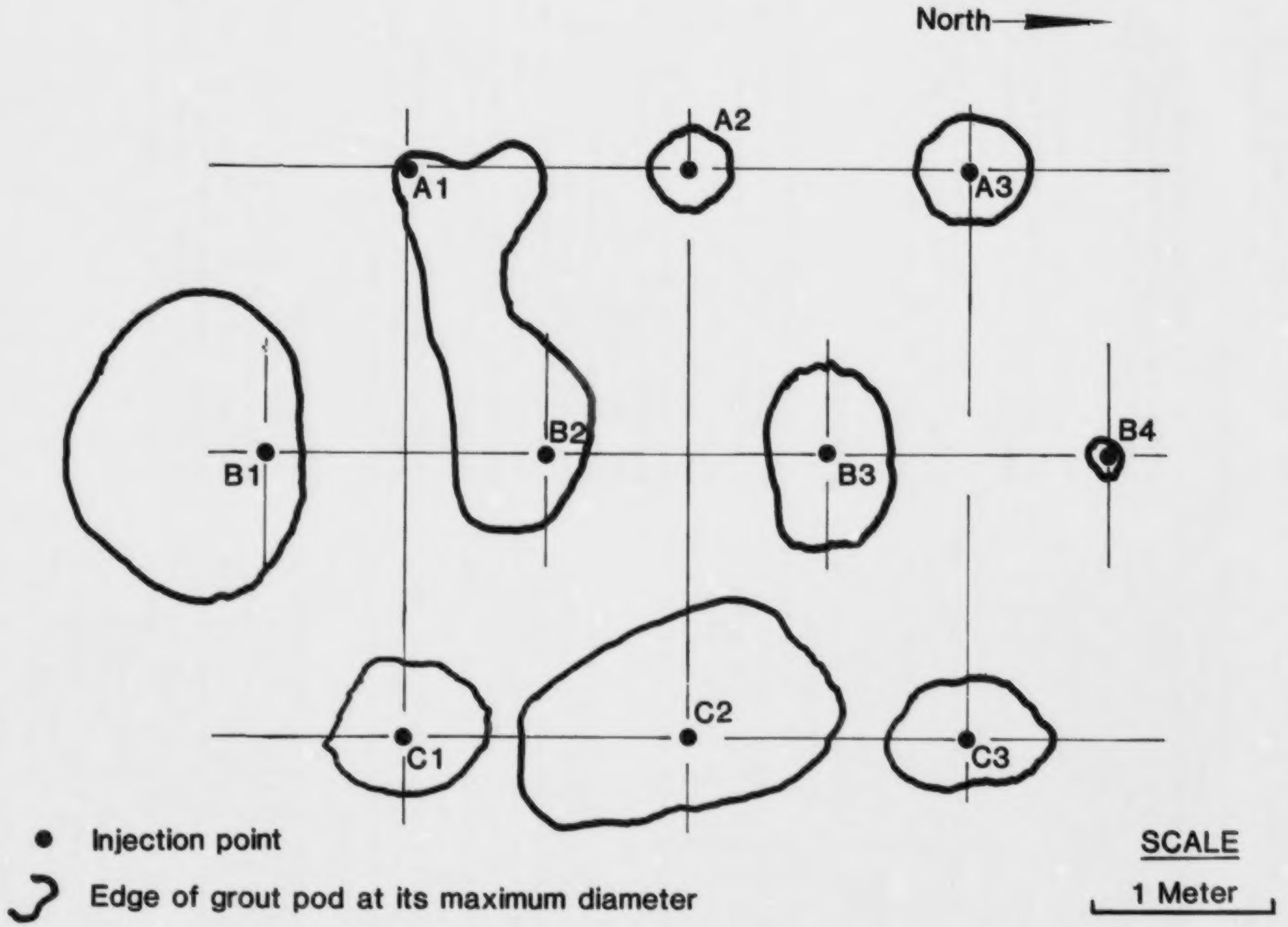
North
A1
A2
A3
B1
B2
B3
B4
C1
C2
C3
● Injection point
Edge of grout pod at its maximum diameter
SCALE
1 Meter

Figure 7. Layout of grout injection holes at the larger-scale test area.

assure that the bentonite would hydrate and swell. A 2.5-cm diameter pipe, 4 m in length with both ends sealed with tape, was inserted in the 5-cm pipe. The lower sealed end of the pipe was pressed into the filter sand. The inner pipe was cemented in place with a mixture of 50 percent portland cement and 50 percent mortar sand. A similar portland cement mixture was used to complete the seal around the outside of the larger 5-cm pipe. A 1.5 cm plastic pipe was run down into the fluid cement/sand mixture and agitated to assure that the thin cement mixture would flow to the points where sealing was needed.

The cement/sand mix was allowed to cure for at least 24 hours before grout was injected. When grout was to be injected, the top of the 2.5-cm pipe was untaped and a 1.5-cm plastic pipe with a solid pointed end was run down inside the 2.5-cm diameter pipe and the lower tape was broken by driving the point 3 to 5 centimeters into the filter sand. This procedure assured that the grout had clear access to the filter sand pocket at the end of the grout pipe. The water table was 2.94 m below ground surface. The injection point was 50 cm above the water table when grout was pumped.

A 30-percent sodium silicate grout similar to that used in the small-scale testing was mixed in batches and injected using a progressive cavity pump. Grout injection continued until refusal was obtained or 760 liters was injected. Refusal was indicated by a rapid pressure rise in the grout pipe followed by stalling the pump or by a "blow-out" of grout to the surface either through the soil or the injection hole annulus.

The injection holes on the outside of the plot were grouted first. The two inside holes (B2 and B3 in Figure 7) were grouted last. Only one hole, B4, was not successfully injected. The grout gelled prematurely on this boring and plugged the grout hose. An adjustment in the grout formulation corrected this problem at subsequent injection points. Samples of grout were taken from each batch to assure that the grout gelled. Food coloring was added to each grout batch to allow solidified grout to be identified. Color could only be used to identify a source when fragments of gel could be recovered intact. The brown and red sand masked the color of the gel in the grout pods.

The grout pods were excavated after approximately 120 days. Figure 7 shows the distribution of the pods over the test plot. Three problems were recognized as the pods were excavated:

a. The grouted sand bodies were often unsymmetrical and ungrouted areas were present between pods. A continuous seal had not formed.

b. Holes larger than those between sand grains (root holes or rootlet holes) were not sealed with grout (Figures 8-9).

c. The coarse-grained sand used under the boreholes to distribute the grout was not cemented (Figure 10).

CONCLUSIONS

The results of the test grouting and excavation indicate that the chemical grout injection did not produce a successful seal. The anisotropic flow of grout in the soil prevented the grout pods from meeting even when large grout takes were observed. The acrylate grout can flow through sand without cementing the particles into a cohesive mass. The silicate grout could not seal small openings in soil even though grout coatings indicated these holes had initially been filled with gel. The coarse sand usually contained no silicate grout. The grout, although initially present, flowed out of or shrank away from the coarse sand pockets. Any coarse sand pocket may represent an unsealable hole in a silicate grout barrier.

ACKNOWLEDGEMENTS

The authors gratefully acknowledge the assistance of Dennis L. Bean, Larry G. Caviness, and Steven A. Houston of the Waterways Experiment Station in this phase of the project. Herbert Pahren is the EPA Project Officer for this investigation.

Figure 8. Photo of rootlet hole present in solidly cemented sand pod.

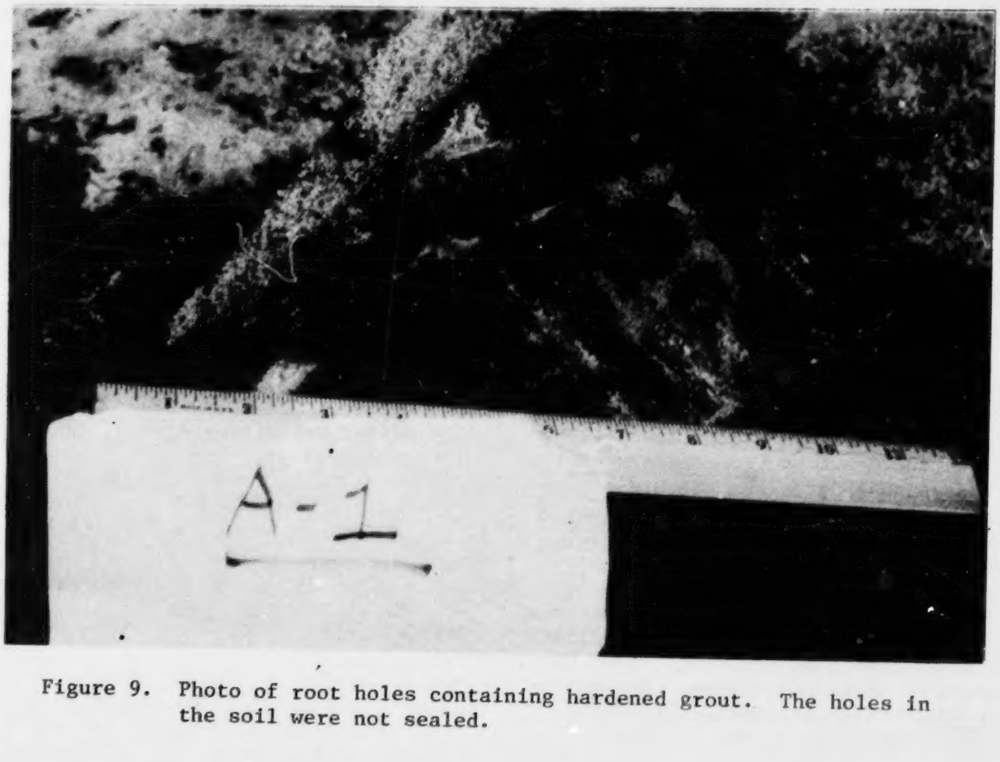

Figure 9. Photo of root holes containing hardened grout. The holes in the soil were not sealed.

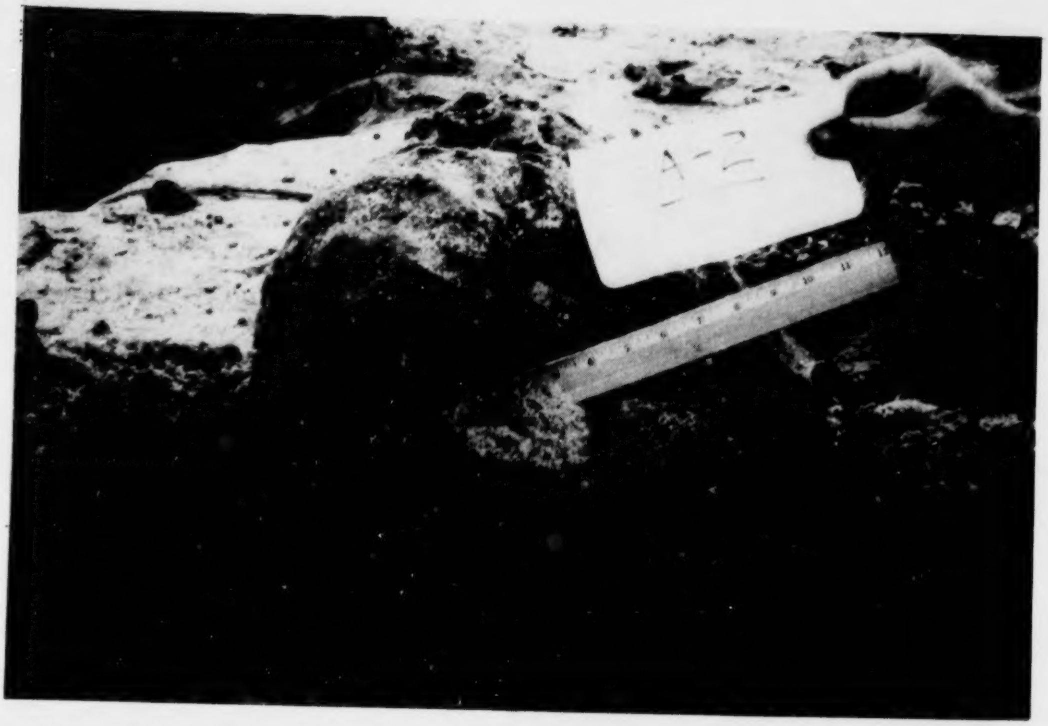

Figure 10. Photo of coarse-grained sand placed at the bottom of the grout-injection hole. Note that the sand was not cemented although the adjacent fine sand was cemented.

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PERMEABLE MATERIALS FOR THE REMOVAL OF POLLUTANTS FROM HAZARDOUS WASTE LEACHATES

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ABSTRACT

In order to better understand the potential effectiveness of permeable treatment systems, four readily available, low-cost permeable materials -- limestone, coal, fly ash, and a soil containing clay -- were tested to determine their ability to remove organic pollutants from two simulated hazardous waste leachates. The capabilities of various sequentially ordered layers of these materials were evaluated with respect to their ability to retain total organic carbon (TOC) and twelve selected priority pollutants. As a result of testing, the most effective ordering of materials was found to be a layer of fly ash, followed by a layer of coal, followed by a layer of limestone.

INTRODUCTION

As the number of hazardous waste sites requiring remedial action increases, technological alternatives that not only enclose sites, but mitigate contamination will become increasingly popular. One such alternative might use permeable materials in systems designed for the retention of pollutants from hazardous waste leachates. These permeable treatment sysems could be designed as vertical treatment walls or horizontal treatment beds. In the former case, the treatment wall would partially or totally encircle a site in a manner similar to a slurry trench, retaining pollutants as leachates moved through the wall. In the latter case, the treatment bed would be used to pretreat pumped leachate prior to its release to an industrial or municipal wastewater treatment plant. These types of permeable treatment systems are described in the handbook, Remedial Action at Waste Disposal Sites (19), but are identified as conceptual rather than developed, state -of-the-art technologies. In order to better understand the utility of the permeable treament systems concept, four readily available, low-cost materials were tested by measuring their effectiveness in retaining organic pollutants from two simulated hazardous waste leachates. The four materials tested were limestone, fly ash, and a soil containing clay.

Prior to testing, a review of the literature indicated that previous work in the area of pollutant retention by permeable materials had been performed by Fuller, et al (4, 5, 6, 7, 8), Korte (13, 14, 15) and Alesii (1) using soils and crushed limestone: by Griffin, et al (9, 10, 11, 12) emphasizing soils: by Chan, et al (2, 3) and Sheith (16) investigating fly ash, clays and other materials; and Spoljaric and Crawford (17, 18) examining greensands. These studies focused on inorganic pollutants and provided a basis for understanding the retention of metals and other inorganic toxic pollutants by soils containing clay, limestone, fly ash, and greensands. Little information was found in the literature regarding retention of organic pollutants by the aforementioned materials.

PURPOSE

The purpose of this study was to test four readily available, low-cost permeable materials -- limestone, coal, fly ash, and a soil containing clay -- for their ability to retain organic pollutants from two simulated hazardous waste leachates. Various combinations of the materials, ordered in sequential layers, were tested, and their interactions with selected organic pollutants were measured so that the individual and collective performance characteristics of the materials could be measured.

APPROACH

The choice of permeable materials was based on their estimated ability to retain pollutant concentrations, as well as their local availability. Natural or waste materials were judged to have an apparent cost advantage over such man-made materials as activated carbon or ion exchange resins. For this reason, readily available, low-cost materials were tested. Coal was obtained from a mine near Stinson, KY, limestone from a local dealer, fly ash from the Miami Fort Power Plant near Harrison, OH, and soil from northern Hamilton County near Cincinnati, OH. The particle size distributions for each of these materials are shown in Figure 1.

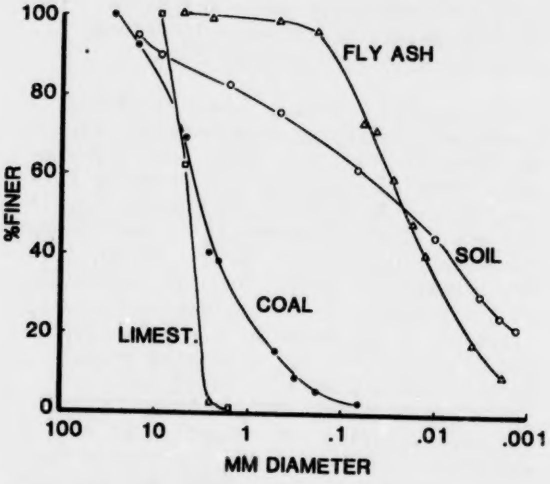


Figure 1. Particle Size Distribution

The materials were placed in 12, 20.3 cm x 20.3 cm cross-sectional columns. Layer thicknesses were chosen for comparative purposes of volume, 20 cm being twice the volume of 10 cm; 6 cm layers were also used mostly for limestone. The total height of all columns was 36 cm. The layer orderings used for both experiment runs are shown in Figure 2. The feed for each run was common to all columns. Porous stones connected to external sample ports were placed between each layer and at the bottom of each column. An illustration of the sampling devices used is shown in Figure 3. These allowed the TOC concentration to be monitored daily as the simulated hazardous waste leachate flowed through the materials in the 12 columns.

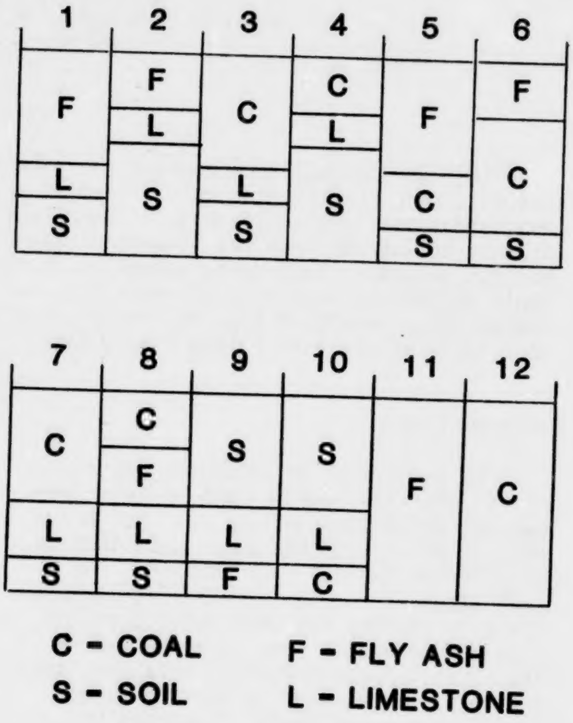


Figure 2. Materials Ordering

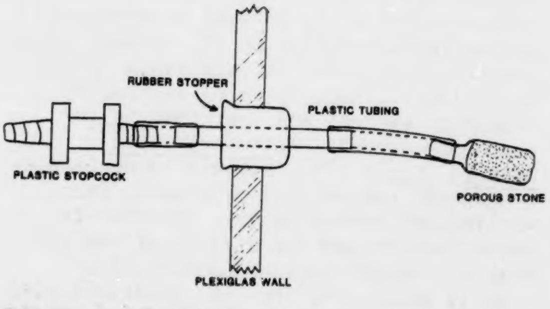


Figure 3. Sample Device

Two different simulated leachates were used in the test. Since organics were of primary concern, total organic carbon (TOC) analysis was used as a measure of their presence in both Runs #1 and #2. Gas chromatographic (GC) analysis for 12 spiked priority pollutants was performed on the effluents from Run #2 to provide an indication of their relative mobilities. The first simulated leachate was comprised of experimental solid waste lysimeter leachate spiked with phenol and dichlorobenzene at 400 mg/L and 80 mg/L, respectively. The TOC level for Run #1 began at 2500 mg/L and decreased to 400 mg/L by the end of the run. Average column feed concentrations based on total feed volume and daily TOC concentrations were 115 mg/L and 1596 mg/l respectively. The simulated leachate for Run #2 was prepared daily and consisted of 0.1N $CaSO_4$, 850 mg/L potassium hydrogen phthalate (KHP), 100 mg/L toluene and the priority pollutants listed in Table 1. The average TOC was 421 mg/l.

TABLE 1. PRIORITY POLLUTANTS IN FEED #2

	Actual Conc. (μg/l)
Bis(2-ethylhexyl) Phthaliate	163
Di-N-Butyl Phthalate	128
1,4 Dichlorobenzene	192
2,4 Dichlorobenzene	109
Ethylbenzene	261
Fluoranthene	129
Isophorone	120
Pentachlorophenol	
Phenanthrene	
Phenol	115
Pyrene	135
Naphthalene	137

Calculations of the retentive capacities were made from the breakthrough curves for each column. Retention was interpreted as the amount of the pollutants, in this case represented by TOC, that remained on a given amount of material (mg/kg dry wt.). Knowing the total TOC input and output, the difference is retained by the permeable materials. Typical examples of breakthrough curves are shown in Figures 4 and 5. The cumulative flow volume and the TOC concentration were normalized to the total flow for each column and the maximum TOC concentration for each experiment. The areas between each curve are proportional to the amount retained by each layer of the column. The total amount of pollutants retained was found by summing the rectangular areas between data points.

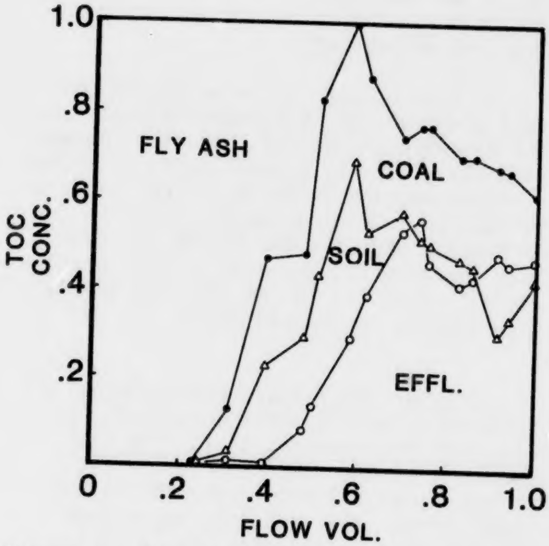


Figure 4. Normalized Breakthrough Curves, Run #1, Column 5

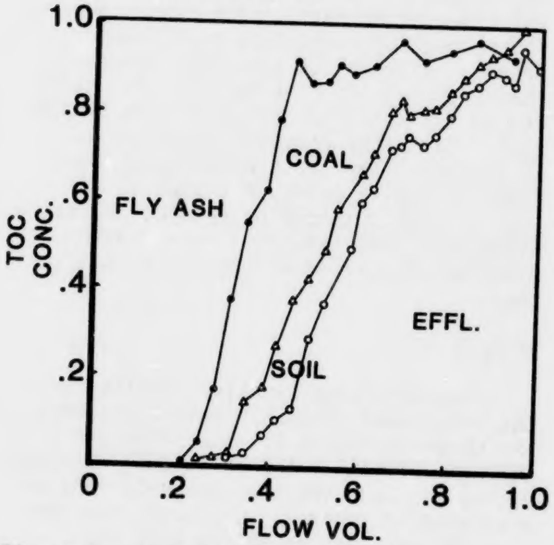


Figure 5. Normalized Breakthrough Curves, Run #2, Column 5

The overall effectiveness of the orderings of materials were compared by ranking four column results: 1) The number of days until 50% breakthrough, 2) the total TOC retained, 3) the linear flow velocity and 4) the calculated permeability coefficient. The first two results are measures of retention, while the last two are indicative of flow. The most effective column then should have both high retention and flow.

PROBLEMS ENCOUNTERED

A problem was encountered with the feed used for the first experimental run. The decrease of the TOC concentration was probably caused by a combination of microbial degradation of the organics and volatilization. This caused decreases in concentration near the end of the run rather than the expected increases. This problem was eliminated in Run #2 when the simulated leachate was spiked daily.

Another problem was the low flow rates in columns 2 and 4 which both contained 20 cm layers of soil. The clay component of the soil reduced the flows through these columns to 200-250 ml/day. Flows in the other columns ranged from 400-750 ml/day. Although this wasn't a problem from an experimental standpoint, the length of the runs, especially Run #2 which lasted 47 days, might cause concern for commercial operations that require answers quickly. It should be noted, however, that the time required for testing is a function of scale and concentration. Since the concentrations tested were higher than those expected at most remedial action sites, the size of the test apparatus should be reduced to facilitate efficient testing time and determination of permeable material effectiveness.

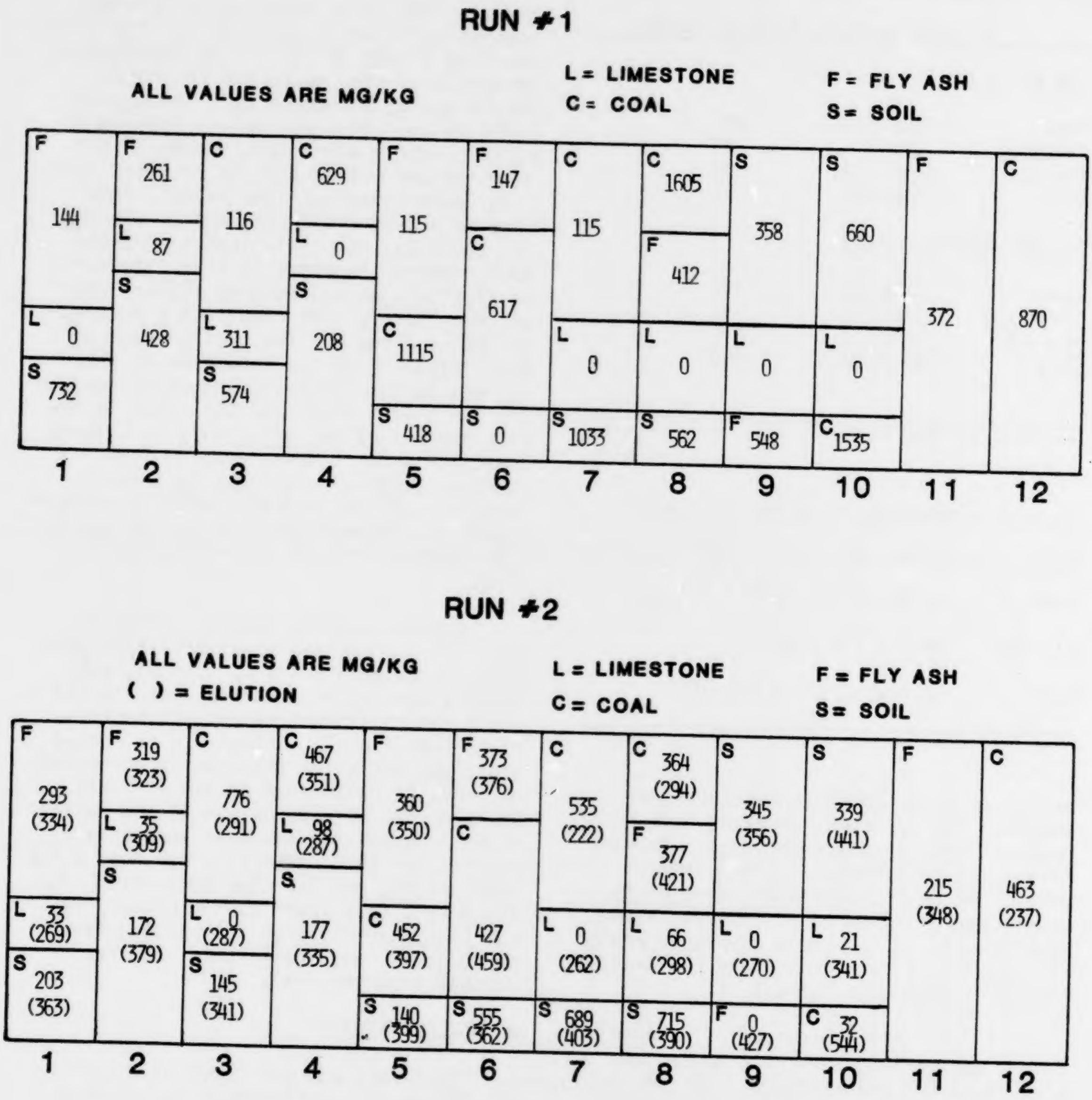
RESULTS

The retentive capacity results for the individual permeable material layers are shown in Figure 6. The range and average values for the pollutants retained during the two simulated leachate runs are presented in Table 2. Coal retained the most TOC per unit weight in both runs. The soil, containing about 20% clay-sized particles retained a large amount of pollutants in Run #1 and slightly less in Run #2. The fly ash retention results are similar for both runs, indicating its retentive capacity may not be related to strong adsorption reactions. Limestone retained very little of the organic pollutants, although it did precipitate iron eluted from the coal. Rust colored layers in the limestone were clearly visible when a coal layer was located above a limestone layer.

Batch elution tests were run using 300 ml of 0.1N $CaSO_4$ and 50 g of the concentration-saturated materials from Run #2. The elution values (mg/kg dry wt.) are included in Figure 6, in parentheses. In the case of fly ash, all the retained TOC was eluted, supporting the contention that fly ash is not a strong adsorber. The coal held as much as half the retained TOC following elution, while the soil elution results were mixed, most samples eluting all the retained TOC with a few holding up to 30%. Thus, coal was the only material exhibiting relatively strong adsorbtion capabilities.

The rate of flow through the layers influences retention by the permeable materials. The porosity of the coal layers allowed rapid movement of TOC. Columns 3 and 7, with 20 cm of coal on the top in Run #1, provided less retention than the coal layers in the other columns. Limiting the flow through the coal by a less permeable layer above it, as in Column 6, appeared to increase the retentive capability of coal. The 20 cm layers of soil in Columns 2 and 4 inhibited flow, while the fly ash in Columns 1 and 5 only moderated the flow. In order to better understand the permeability of the materials used, permeameter tests were run and the results are shown in Table 3 for coal, fly ash, and soil. Limestone was not tested in the permeameter since it was a uniformly distributed granular material that was not expected to effect liquid flow rates when in combination with the other materials.

The results of the GC analyses for Run #2 are presented in Table 4. The majority of the occurrences of the twelve priority pollutants in the effluents were at lower levels than in the simulated leachate feed. Generally, even when a compound was detected at a concentration greater than 100 μg/l in a sample, it



RUN #1
ALL VALUES ARE MG/KG
L = LIMESTONE
C = COAL
F = FLY ASH
S = SOIL
F 144
L 0
S 732
F 261
L 87
S 428
C 116
L 311
S 574
C 629
L 0
S 208
F 115
C 1115
S 418
F 147
C 617
S 0
C 115
L 0
S 1033
C 1605
F 412
L 0
S 562
S 358
L 0
F 548
S 660
L 0
C 1535
F 372
C 870
1
2
3
4
5
6
7
8
9
10
11
12
RUN #2
ALL VALUES ARE MG/KG
( ) = ELUTION
L = LIMESTONE
C = COAL
F = FLY ASH
S = SOIL
F 293 (334)
L 33 (269)
S 203 (363)
F 319 (323)
L 35 (309)
S 172 (379)
C 776 (291)
L 0 (287)
S 145 (341)
C 467 (351)
L 98 (287)
S 177 (335)
F 360 (350)
C 452 (397)
S 140 (399)
F 373 (376)
C 427 (459)
S 555 (362)
C 535 (222)
L 0 (262)
S 689 (403)
C 364 (294)
F 377 (421)
L 66 (298)
S 715 (390)
S 345 (356)
L 0 (270)
F 0 (427)
S 339 (441)
L 21 (341)
C 32 (544)
F 215 (348)
C 463 (237)
1
2
3
4
5
6
7
8
9
10
11
12

Figure 6. Retention and Elution Results, Runs 1 and 2

TABLE 2. AVERAGE RETENTION CAPABILITIES

	Range(mg/kg)	Average(mg/kg)
Run #1 (2500-400 mg/l TOC)		
Coal	(1605-115)	825
Limestone	(311-0)	50
Fly ash	(548-115)	286
Soil	(1033-0)	397
Run #2 (421 mg/l TOC)		
Coal	(776-32)	440
Limestone	(98-0)	32
Fly ash	(377-215)	277
Soil	(715-140)	348

TABLE 3. PERMEABILITY RESULTS (cm/sec)

Coal	$1.8 - 5.6 \times 10^{-4}$
Fly ash	$5.3 - 7.0 \times 10^{-5}$
Soil	$1.7 \times 10^{-4} - 3.5 \times 10^{-5}$

would not be detected in the subsequent sample. The detected compounds appeared predominately in discrete cases. Phenanthrene, fluoranthene, dichlorophenol, pyrene and pentachlorophenol were either undetected or appeared at concentrations less than 20 μg/l. These compounds would be less likely to migrate than the other compounds listed in Table 1.

The four criteria used for evaluating column performance were: 1) the number of days to 50% breakthrough, 2) the total TOC retained, 3) the linear flow velocity, and 4) the calculated permeability coefficient. As previously mentioned, the first two results are measures of retention, while the last two results are indicative of flow. The best performing colcolumn then would have both high retention and high flow . The overall rankings for performance were found by assigning numerical values (1 through 12) to each of the columns for each of the four results. For example, in Run #1 Column 12 retained the most TOC so it was given a "1" ranking, while Column 7 retained the least and received a rank of "12." The four ranked results were then added and the totals ranked again, 1 through 12. The orginal rankings were also multiplied together; these products werre then ranked. The additive and multiplicative rankings were then added together. The overall rankings were determined from these numbers. The additive and multiplicative methods were combined in order to afford better resolution when two or more columns were closely ranked. The results are presented in Tables 5 and 6. The overall rankings are indicative of the responses of the columns to the simulated leachates used in the two runs. Since the soil used in the test hindered flow, especially in Columns 2 and 4, these columns received lower rankings despite long times to breakthrough. Columns 3 and 7, with 20 cm coal layers, performed poorly in Run #1 due to high permeability and possible channelling.

Considering the results of Run #2, the most appropriate layer orderings were found in Columns 5 and 6, with fly ash above a coal layer. Ranked next was Column 3, which did not repeat its performance from Run #1. Column 8 also ranked high, and exhibited the least occurrences of priority pollutants in its effluent. Based on the results of Run #2 and indications from Run #1, the most appropriate ordering of these materials would be a top layer of fly ash for flow moderation, followed by a coal layer as the primary retaining material, then a limestone layer to adjust pH and precipitate the iron leached from the coal.

ACKNOWLEDGEMENTS

This work was performed as a part of the University of Cincinnati's contract to operate the US EPA's Test and Evaluation (T&E) Facility. The author gratefully thanks Yu-Lin Liu for her considerable help in running the tests. The Technical Project Monitor and provider of guidance was Jon Herrmann.

TABLE 4. PRIORITY POLLUTANT GC ANALYSES RESULTS

Column	Number Compounds Detected	Samples 7100 μg/l	Compounds >100 μg/l
1	30	2	Dibutylphthalate, Phenol
2	25	1	Phenol
3	21	1	Bis(2-ethylhexyl) Phthalate
4	13	1	Phenol
5	17	1	Bis(2-ethylhexyl) Phthalate
6	16	3	Phenol, Bis(2-ethylhexyl) Phthalate
7	20	5	Phenol, 1,2 Dichlorobenzene, Bis(2-ethyl...)
8	10	0	
9	21	5	Bis(2-ehtylhexyl) Phthalate, Phenol, Isophorone, Dibutylphthalate
10	14	3	Napthalene, Dibutylphthalate, Ethylbenzene
11	14	4	Bis(2-ethyl...), Dibutylphthalate, Ethylbenzene
12	13	3	Bis(2-ethyl...), Dibutylphthalate

TABLE 5. RUN #1 COLUMN PERFORMANCE DATA

Column	Total Flow (l)	Total mg Retained	Days to 50% Breakthrough	Linear Velocity (cm/day)	Calculated K (cm/day)	Overall Ranking
1	7.18	6377	16	19.3	.78	8
2	6.66	7839	18	17.2	.73	6
3	6.79	5709	9	10.2	.96	11
4	3.93	5247	19	7.5	.41	10
5	8.95	7299	11	18.8	.98	4
6	7.74	6103	12	12.4	.85	9
7	6.47	5177	7	9.2	.91	12
8	9.22	10493	11	21.4	1.01	2
9	6.21	6567	12	27.8	.68	7
10	9.23	11683	18	23.5	.95	1
11	8.08	7900	19	9.4	.84	5
12	8.92	12274	7	22.2	.97	3

TABLE 6. RUN #2 COLUMN PERFORMANCE DATA

Column	Total Flow (l)	Total mg Retained	Days to 50% Breakthrough	Linear Velocity (cm/day)	Calculated K (cm/day)	Overall Ranking
1	15.86	4448	26	34.5	.84	5
2	12.38	3857	39	21.7	.52	9
3	25.78	7064	30	13.8	1.12	3
4	11.86	4310	47	8.8	.50	8
5	23.99	6146	16	60.5	1.47	2
6	24.02	7525	37	41.2	1.01	1
7	26.12	6810	19	14.8	1.10	7
8	28.40	7006	20	16.0	1.19	4
9	14.35	3796	13	23.3	1.36	11
10	16.12	3911	14	25.9	1.00	12
11	17.00	4374	28	31.1	.77	6
12	29.44	6533	15	8.2	1.23	10

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MECHANISMS OF CONTAMINANT MIGRATION THROUGH A CLAY BARRIER--CASE STUDY, WILSONVILLE, ILLINOIS

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ABSTRACT

Routine monitoring of a hazardous waste disposal facility at Wilsonville, Illinois revealed that organic contaminants were migrating 100-1000 times faster than predicted. A detailed hydrogeologic investigation of the site revealed that the local groundwater flow and gradient were dominated by a 45-foot high coal mine refuse pile that causes radial flow from the pile through the trenches. Measurements of hydraulic head indicated that the vertical flow was much less significant than the lateral component of groundwater flow. Results of hydraulic conductivity tests and geologic investigations lead to the conclusion that the higher-than-predicted migration rates could be accounted for by the differences between laboratory and field measurements of hydraulic conductivities. These differences appear to be primarily due to the inability of small laboratory specimens, whether "undisturbed" or recompacted, to simulate the flow through relatively large joints or partings present in natural materials.

The distribution of contamination in the groundwater was clearly related to the flow patterns. Time series sampling of monitoring wells in slowly recharging formations showed that concentrations of volatile organics varied widely through the recovery period.

INTRODUCTION

Laboratory studies have shown that organic chemicals can increase the hydraulic conductivity of compacted clay soils; several field studies have documented instances where field-measured hydraulic conductivity of compacted clay liners exceeded the laboratory-measured values used to design the facility. These and other studies have resulted in concern about the potential failure of clay barriers to control release of leachate from landfills and surface impoundments.

Interactions between clay, percolating water, organic solvents, and contaminants can result in rates of migration greater than expected. Anderson, Brown, and Green (2), Acar, Oliveri, and Field (1), and Foreman and Daniel (7) have reported that hydraulic conductivities of soils and clay could be greatly increased when exposed to various organic solvents. Bridley, Wiewiora, and Wiewiora (3) and Griffin et al. (10), found that clay minerals that expand in water can undergo relative collapse when exposed to organic solvents. Murray and Quirk (15) have shown that dry, pressed pellets of non-expanding clays will expand when solvated by various solvents in direct proportion to the dielectric constant of the solvent. These studies support a mechanism for producing higher hydraulic conductivities in which clay materials exposed to water, which has a relatively high dielectric constant, shrink when exposed to organic solvents with a lower constant.

Failure of clay barriers may also result from measurement errors during design or construction. Daniel (5) reported four case studies of clay-lined impoundments where unexpectedly high leakage rates were traced to underestimation of hydraulic conductivity because laboratory measurements were used; field-measured hydraulic conductivities were found to be much greater.

Failure of some clay barriers to prevent leachate migration may result from natural as well as man-induced conditions. For example, evaluators of potential landfill sites in unconsolidated glacial deposits often overlook the existing natural joint system. Contaminated aqueous wastes and organic solvents can move along these routes. Additionally, potential clay-solvent interactions that can enhance these existing natural joints or planar structures have usually been ignored during landfill and surface impoundment siting and design.

The case study reported here began as an outgrowth of a 1982 court order to exhume and remove the hazardous wastes buried at a disposal facility at Wilsonville, Illinois. The 130-acre landfill operation was a trench-and-fill procedure that basically relied on natural attenuation of contaminants by a clay-containing till deposit native to the site. Routine monitoring of the site revealed that organic contaminants were migrating 100 to 1000 times faster than predicted. The Illinois State Geological Survey, supported by the U.S. Environmental Protection Agency, the Illinois Environmental Protection Agency, and the site owner, SCA Services, Inc., began a study to determine the cause or causes of contaminant migration at the facility. Two obvious questions were posed: (1) Why were these organic contaminants migrating faster than predicted, and (2) what were the implications for land disposal of similar wastes at other sites?

The background events, overall site characteristics, and project description have been published previously (9, 13, and 21). The geology of the site has been described by Follmer (6) and by Griffin et al. (10).

PURPOSE

The purpose, objectives, and approach for the project have been described in detail in earlier publications (9 and 13).

This paper describes hydrogeologic conditions at the site and how they affect the distribution of contaminants in the groundwater. To determine the mechanism responsible for the higher-than-expected migration rates of organic contaminants at the site, hydraulic conductivity was measured by several methods in the field and in the laboratory, and the effect of joints in the till on the measured values was examined. The results of these measurements are particularly significant because relatively few data have been published that compare laboratory- and field-determined hydraulic conductivity values from actual disposal sites.

This paper also discusses results of time series sampling of volatile organics from monitoring wells finished in fine-grained earth materials. This sampling work was conducted to determine the effect of sample collection procedures on the amounts of volatile organics detected in water samples.

RESULTS

The work described in this paper is currently in progress. The results presented should be considered as tentative and subject to possible reinterpretation.

Hydrogeologic/Geochemical Studies

Hydrogeologic and geochemical investigations of the site have involved the completion of 17 piezometer/monitoring well nests (nests A to K, profiles V and W in Figure 1), with two to nine piezometers and monitoring wells per nest. The details of the design, construction, and experimental plan were published previously (9, 10, and 13).

The piezometers were used initially for in-situ hydraulic conductivity tests at various depths. After the water levels stabilized, these piezometers were used to establish the long-term piezometric surface and, in turn, the

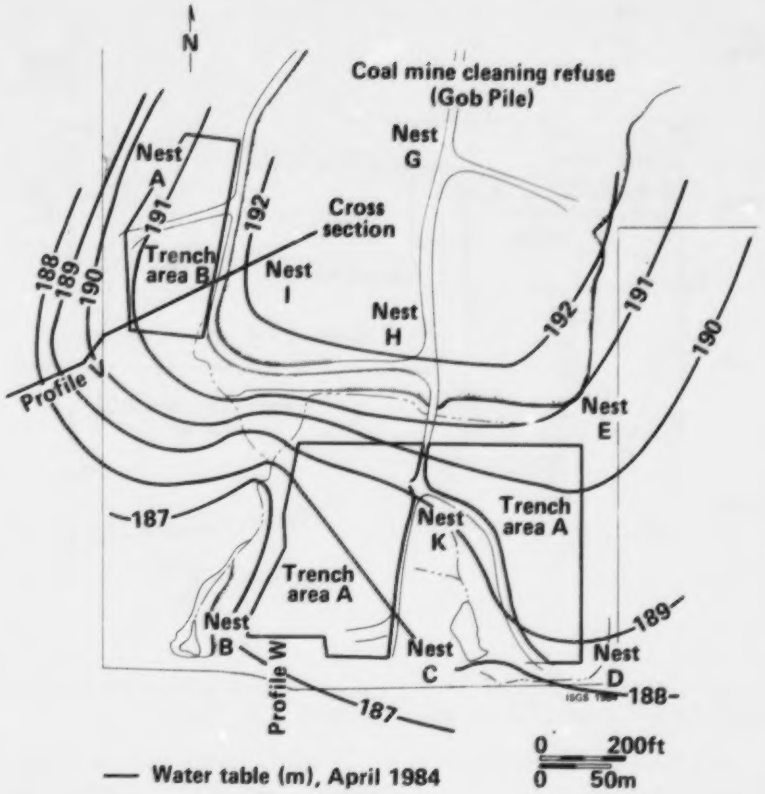
N
Coal mine cleaning refuse
(Gob Pile)
Nest G
Nest A
191
192
Cross section
Trench area B
Nest I
Nest H
188
189
190
Profile V
192
191
190
Nest E
187
Nest K
Trench area A
Trench area A
Nest B
Profile W
Nest C
Nest D
189
188
187
0 200ft
0 50m
Water table (m), April 1984

Figure 1. Site map showing: water table elevation (m) in April, 1984; locations of trenches, wells and cross section

hydraulic gradient and flow across the site. The elevation of the water table measured in all wells and piezometers at the site, including SCA wells in April, 1984, is shown in Figure 1. The effect of the coal refuse pile on the shallow groundwater flow pattern is evident. A groundwater mound is present beneath the coal refuse pile, resulting in shallow groundwater flow in a radial pattern toward trench area B to the west and trench area A to the south. Some disposal trenches in each area were apparently excavated below the water table; however, the trenches may have remained relatively dry during the disposal operation due to the very low hydraulic conductivity of the surrounding glacial till.

Figure 2 is a cross section of the site along the line shown on Figure 1. This cross section shows the sequence of geologic materials in this area; these materials have been described in detail by Follmer (6) and Griffin et al. (10). The cross section also shows the position of the water table and equipotential lines as measured in April, 1984. Measurements of hydraulic head in wells at different depths in each cluster of wells in Profile V and at Nest I indicated a downward vertical hydraulic gradient and groundwater flow. However, these were much less significant than the lateral component of groundwater flow dominant in each geologic unit because of the lateral hydraulic gradient.

Hydraulic Conductivity Tests

The determination of hydraulic conductivity at the site was an

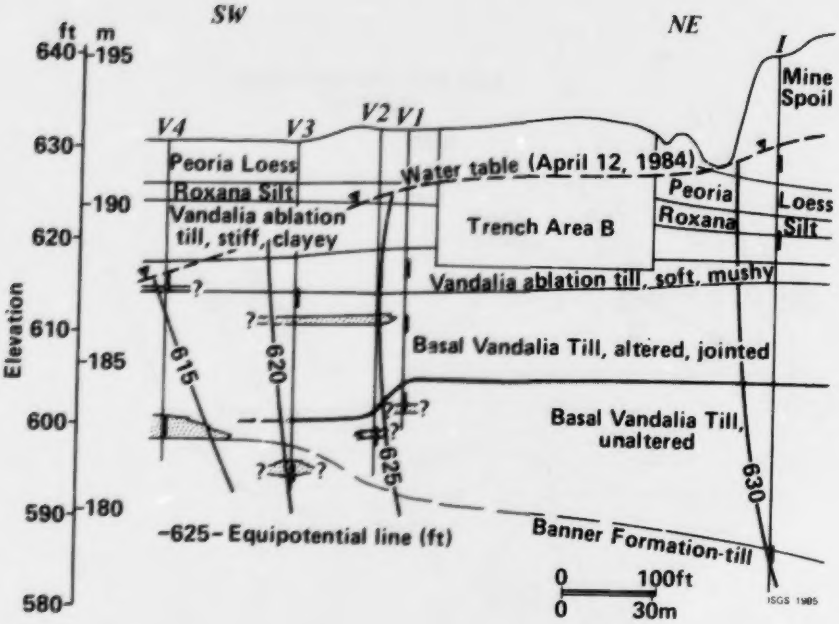
SW
NE
ft m
640
195
630
190
620
610
185
600
590
180
580
Elevation
V4
V3
V2
V1
I
Mine Spoil
Peoria Loess
Roxana Silt
Vandalia ablation till, stiff, clayey
Water table (April 12, 1984)
Trench Area B
Peoria
Roxana
Loess
Silt
Vandalia ablation till, soft, mushy
Basal Vandalia Till, altered, jointed
Basal Vandalia Till, unaltered
615
620
625
630
?
-625- Equipotential line (ft)
Banner Formation-till
0 100ft
0 30m
ISGS 1985

Figure 2. Cross section from Profile V through Trench Area B to the gob pile showing equipotential lines of groundwater flow.

essential part of this project. Therefore, several methods were used to measure it.

Laboratory values of hydraulic conductivity were determined from tests of undisturbed and recompacted soil cores, using a Harvard-type miniature permeameter. This method is described in detail in Herzog and Morse (11). Additional values were available from engineering reports about the site (12 and 8). Field analyses included slug tests analyzed by the method of Cooper, Bredehoeft, and Papadopulos (4) and by a method proposed by Nguyen and Pinder (16). Recovery tests were performed on monitoring wells that were pumped for the purpose of well development, using the technique described by Todd (22).

Field tests were performed on both vertical and angle holes. In addition to the vertical piezometers installed at each nest shown in Figure 1, angle piezometers were also installed at nests A, B, E and K to attempt to measure the influence of vertical joints on hydraulic conductivity.

Angle holes were drilled at a 45° angle. The cased interval was backfilled by pumping grout from the bottom of the casing to the ground surface rather than from the intervals of expanding concrete and bentonite described in earlier reports (9 and 13). All other design and construction details were identical to those described for the vertical piezometers. Construction details of the monitoring wells and vertical piezometers are shown in Figure 3. Figure 4 illustrates that the angle holes have a much greater chance of intersecting vertical fractures because the holes are longer and cover a greater lateral area.

Slug tests were originally designed to be analyzed by the method proposed by Cooper, Bredehoeft and Papadopulos (4), and the type curves published by Papadopulos, Bredehoeft and Cooper (18). This method was chosen because it assumed boundary conditions that could reasonably be met in the field.

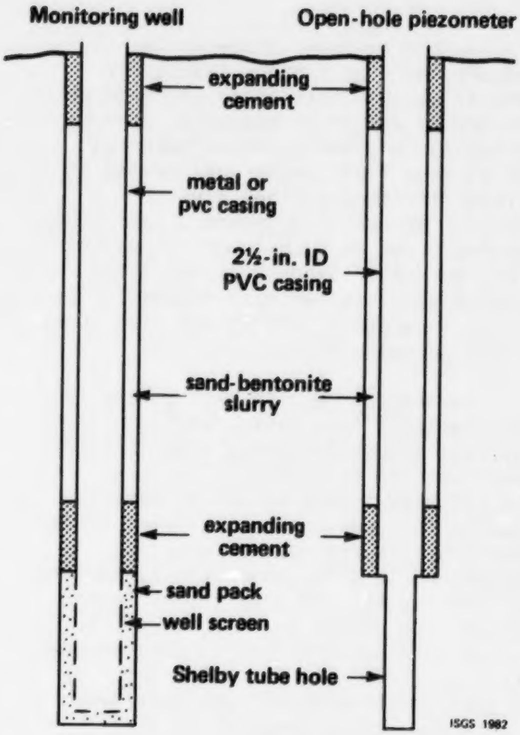
Monitoring well
Open-hole piezometer
expanding cement
metal or pvc casing
2½-in. ID PVC casing
sand-bentonite slurry
expanding cement
sand pack
well screen
Shelby tube hole
ISGS 1982

Figure 3. Construction details for monitoring wells and vertical piezometers.

All slug test data were reanalyzed using the method of Nguyen and Pinder (16). The method is a direct calculation method that uses data obtained during the early part of the slug test (usually within the first half-hour). Our slug tests were not originally designed for analysis by this method, and many of the data were less than ideal for analysis by this method because few early measurements had been taken. Additional error may have resulted from using discrete measurements obtained by steel tape or electronic water level meter instead of having continuously recorded water levels.

The Nguyen and Pinder method is designed to consider problems involving partially penetrating wells in aquifers for which the short-term effects of a water table or leakage from a confining layer can be ignored. It appears to be appropriate for analyzing tests in materials of moderate to low hydraulic conductivity. The boundary conditions (except for full aquifer penetration) and parameters to be measured are the same as in the Cooper, Bredehoeft, and Papadopulos method.

The third field method used was the recovery test (22). The recovery test, often used after an aquifer pumping test, applies to unsteady radial flow in a confined aquifer, if the well fully penetrates the aquifer and well storage is negligible. For this study, the recovery test was used to analyze all monitoring wells that were pumped for development.

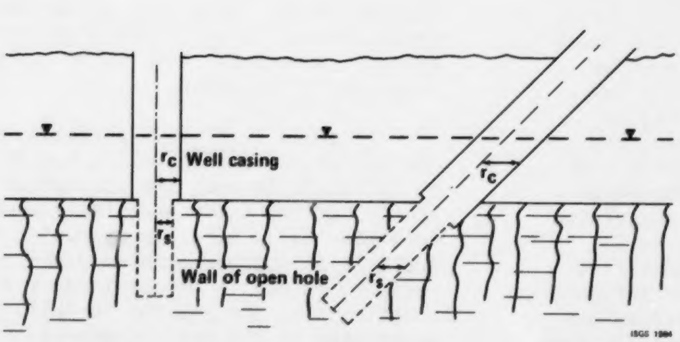
r_c Well casing
r_c
r_s
Wall of open hole
r_s
ISGS 1984

Figure 4. Parameters to be measured for slug test analysis.

Table 1 summarizes all the laboratory and field hydraulic conductivity data collected to date. Data for many individual measurements have been reported previously (11). Field data are presented as the geometric means because hydraulic conductivity values in field soils are generally acknowledged to be log-normally distributed (17 and 19), so arithmetic averaging would not be appropriate (14). For some formations, the data had a range of about three orders of magnitude. Because of the heterogenity of glacial tills, this variability is not unexpected.

The number of values averaged ranged from 1 for the Banner Formation-bedrock contact and some tests on the Sangamon soil, to 13 for recovery tests in the weathered, basal Vandalia Till. The laboratory values are reported as ranges because they include data reported by consultants who did not report individual values (8 and 12).

As reported in Table 1, the range of values determined in the laboratory for tests of each type of sample, disturbed and undisturbed, is small. The undisturbed samples yielded higher values of hydraulic conductivity than the disturbed samples for soft sediments, but the reverse is true for the hard tills.

Whereas the field data showed a wide range of values, the geometric means calculated from the data from the three methods were nearly equal. The most divergent value was found in the Sangamon Soil Bt horizon for which there was only one slug test site and one recovery test site and these were widely separated (1/2 mile apart). The unaltered basal Vandalia Till also shows disagreement between slug tests and recovery tests. For this unit, the higher mean value from the recovery tests can be explained by noting that the recovery tests were performed on monitoring wells and that three of the nine wells used for recovery test analysis were finished in zones that included a sand lens in their screened interval, thus yielding higher values than from wells or piezometers finished in the matrix material.

On the basis of these tests, it appears that the various laboratory methods used in this study can produce comparable values of hydraulic conductivity for a given geologic material. The various field techniques tested at Wilsonville also yielded similar values. However, in general, laboratory methods produced much lower values than field methods. Laboratory and field results were similar only between undisturbed laboratory samples and slug tests of the Sangamon Bt horizon.

Laboratory results were generally less variable than field results. This may have occurred because fewer samples were run, but is probably due to a natural bias toward selecting cohesive samples for undisturbed analysis; crumbling or fractured samples were not chosen for testing, thus leading to more uniform results and bias toward lower values. Recompaction of samples also yields more uniform results because the original structure, including any natural joints present, are destroyed during the sample preparation steps. This suggests that laboratory measurements of hydraulic conductivity should not be substituted for field measurements unless caution is used in their interpretation.

The biggest disadvantage of the Cooper, Bredehoeft, and Papadopulos method is the length of time necessary to obtain results for materials that have a very low hydraulic conductivity. Some tests in this study ran for more than a year before water levels stabilized. Therefore, error may be introduced by seasonal changes in regional water levels and by barometric fluctuations. A second problem results from the assumption that wells fully penetrate the aquifer.

Recovery tests seem a reasonable choice for monitoring wells, but it may be difficult to meet the assumption of a steady pumping rate, especially when the formation has a very low hydraulic conductivity. Another problem with the recovery method is its basis on the assumption that the aquifer is fully penetrated by the well screen--an assumption that is not usually valid. Constructing monitoring wells to meet this latter condition usually reduces their effectiveness as monitoring wells. However, the test can be performed relatively quickly as part of a well development program to give additional information on the geologic unit that the well monitors.

The Nguyen and Pinder method offers several advantages over the other field methods. This method was designed for partially penetrating wells--the most common type of well construction. In addition, the test usually takes less than a day to run; therefore, it is not influenced by seasonal water level changes, and barometric fluctuations are minimized. The biggest disadvantage appears to be that small errors in water-level measurements will affect the results of this method more than those from the other two field methods. However, these errors can be minimized by using a sensitive, continuous water-level recorder.

Geometric mean values of hydraulic conductivity from the angle tests were greater than the corresponding values from the vertical tests for both types of slug test analyses. This was not necessarily true for all piezometer pairs. The mean difference ranged from less than a factor of two for the ablation zone to more than an order of magnitude for lower basal till. This difference was expected because angle holes should intersect more of the predominantly vertical fractures (Figure 4). Although these results are interesting, they are not conclusive because the theory for analyzing angle piezometers has not yet been fully developed and the statistical significance of the differences between results for vertical and angle-holes has not been determined.

Although the Nguyen and Pinder method for the analysis of data from slug tests seems to be the best of the three field methods tested, the recovery test and the Cooper, Bredehoeft, and Papadopulos method of slug test analysis should not be abandoned. The recovery test is the only one of these tests that can be performed on individual monitoring wells after pumping. The Cooper, Bredehoeft, and Papadopulos method could be used to supplement the Nguyen and Pinder method because data that cannot reasonably be analyzed using one slug test method may be analyzed by the other.

Contaminant Migration

The hydraulic conductivity tests could explain the rate of contaminant migration. However, it is equally important to determine the pattern of contamination. Therefore, a separate set of monitoring wells was constructed to obtain water samples for chemical analyses at each of the nests and profiles (Figure 1). Cores from these borings were analyzed chemically for organic and inorganic constituents. The results, described in Griffin et al. (10), indicated that the zones of highest contaminant concentration were in the soft ablation till and the weathered, jointed basal till (Figure 5). The results of in-situ field tests (Table 1) indicated that the hydraulic conductivities of the soft ablation zone and weathered basal till are greater than those of the underlying or overlying units; this probably accounts for the higher concentrations in the soil and apparently higher migration rates in this zone.

The extent of contaminant migration in the groundwater flow system at profile V is illustrated in Figure 5, which shows the concentrations of trichloroethylene (μg/L) in groundwater in September, 1984. The distribution of contamination in this cross section is clearly related to the measured groundwater flow patterns in Figure 2. However, the highest levels of contamination were found in wells finished in sand lenses within the upper part of the unweathered, basal till zone. Contaminants from Trench Area B introduced into the soft ablation till and jointed basal till must be able to move downward into the unweathered basal till through interconnected joints or sand lenses. The extent of contaminant migration in this area is significantly affected by the strong lateral component of groundwater flow away from the trench area.

Table 1. Summary of Hydraulic Conductivity Data

		Ranges of Laboratory Data		Geometric Means of Field Data		
				Slug Tests		
MATERIAL	Orientation	Recompacted Samples (cm/sec)	Undisturbed Samples (cm/sec)	Cooper,et.al. Method (cm/sec)	Nguyen & Pinder Method (cm/sec)	Recovery Test (cm/sec)
Peoria Loess	Vertical	$3.3x10^{-8}$ - $3.6x10^{-8}$	$7.5x10^{-8}$ - $1.1x10^{-6}$			
Peoria Loess/gob Contact	Vertical					$4.8x10^{-5}$
Vandalia Till Ablation Zone, Sangamon Bt Horizon	Vertical	$6x10^{-10}$ - $1.2x10^{-9}$	$1.6x10^{-7}$	$1.3x10^{-7}$	$1.2x10^{-7}$	$2.0x10^{-6}$
	45° angle				$6.4x10^{-7}$	
Vandalia Till Lower Ablation Zone	Vertical	$2.6x10^{-8}$ - $3.5x10^{-8}$	$5.8x10^{-7}$ - $1.9x10^{-6}$	$3.8x10^{-5}$	$1.9x10^{-5}$	$1.6x10^{-5}$
	45° angle			$7.5x10^{-5}$	$2.4x10^{-5}$	
Base of Ablation Zone/top of weathered Vandalia	Vertical					$1.3x10^{-6}$
Weathered basal Vandalia Till	Vertical 45° angle		$1.2x10^{-8}$ - $7.4x10^{-8}$	$2.4x10^{-6}$ $9.5x10^{-6}$	$2.3x10^{-6}$ $1.2x10^{-5}$	$4.0x10^{-6}$
Unaltered basal Vandalia Till	Vertical 45° angle	$8.9x10^{-9}$ - $2.7x10^{-8}$	$3.3x10^{-9}$ - $5.1x10^{-9}$	$1.8x10^{-7}$ $2.5x10^{-6}$	$8.4x10^{-8}$ $9.4x10^{-7}$	$6.8x10^{-7}$
Vandalia Till/Banner Formation contact	Vertical					$9.9x10^{-7}$
Banner Formation/ bedrock contact	Vertical				$1.1x10^{-7}$	$1.7x10^{-7}$

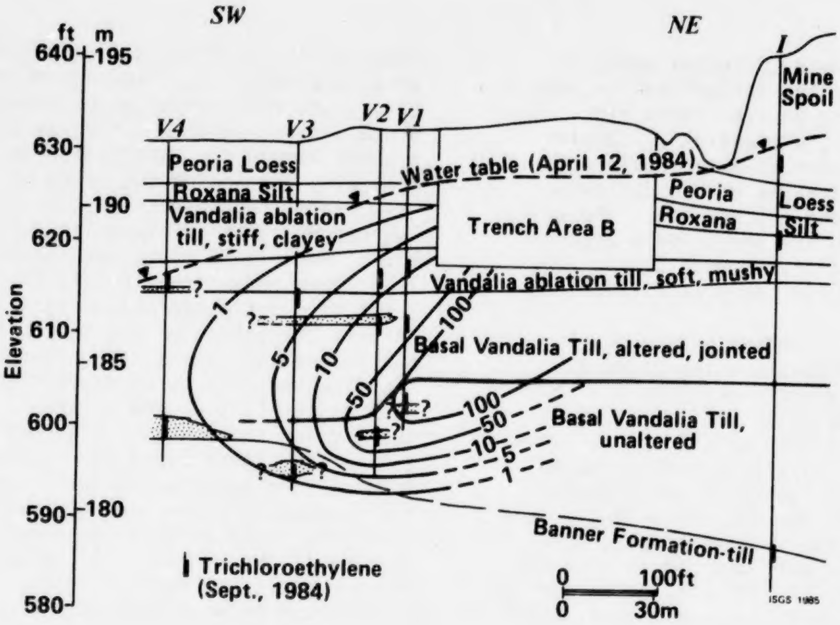
SW
NE
ft
m
640
195
630
190
620
610
185
600
590
180
580
Elevation
V4
V3
V2
V1
I
Mine Spoil
Peoria Loess
Roxana Silt
Vandalia ablation till, stiff, clayey
Water table (April 12, 1984)
Trench Area B
Peoria
Roxana
Loess
Silt
Vandalia ablation till, soft, mushy
Basal Vandalia Till, altered, jointed
Basal Vandalia Till, unaltered
1
5
10
50
100
?
Banner Formation-till
Trichloroethylene (Sept., 1984)
0
100ft
0
30m
ISGS 1985

Figure 5. Cross section from Profile V through Trench Area B to the yob pile showing distribution and Trichloroethylene (ppb) in groundwater.

Volatile Organic Sampling in Fine-Grained Material

A protocol has not been established for collecting water samples from monitoring wells finished in fine-grained materials with slow recharge rates for analysis of the water's volatile organic content. Schuller, Gibb, and Griffin (20) have recommended evacuation of 4 to 6 well volumes to purge the well of stagnant water prior to collection of samples. However, this is not possible for wells with very slow recharge rates; most wells at the Wilsonville site could be pumped down to the top of the well screen after removing less than 2 well volumes. Thus, the question arises as to what impact the well recovery time has on the chemical composition of water samples collected for analysis.

To address this question, sets of time-series water samples were collected at several wells. Representative results obtained for o-xylene from three wells are shown in Figure 6. The results clearly indicate that changes in o-xylene concentration occur as a function of well recovery time.

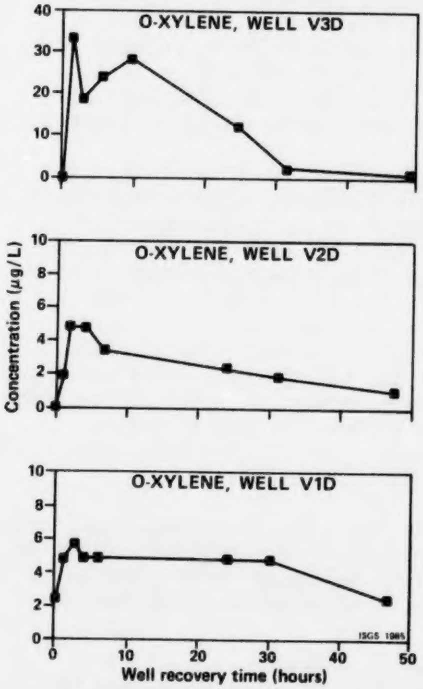

Figure 6. Volatile Organic Concentration vs. Time

Samples were collected using a double check valve Teflon^R bailer with a bottom draining device (Timco Mfg., Prairie Du Sac, Wisconsin)*. Samples were drained into 40-mL volatile organic analysis (VOA) vials with a Teflon^R-faced septum and a screw cap. The vials were placed on ice immediately after collection and transported to the laboratory within 48 hours of collection.

Samples collected before purging the well of stagnant water (time 0) contained o-xylene concentrations that were below detection limits at two of the three wells for which data are shown in Figure 6. This is probably due to volatilization losses of the compound resulting from the water standing in the well. After the initial samples were collected, the well was pumped down to the top of the 2-foot well screen with a bladder-pump; the pump was then removed from the well. Samples were then collected by bailer at intervals of approximately 1, 2, 4, 6, 24, 31, 48, 124, 290, and 460 hours. Maximum concentrations were obtained after approximately 2-8 hours of recharge. Concentrations fell to low values between 24 and 48 hours of recharge. Concentrations of o-xylene in all samples collected after 124, 290, and 460 hours were below detection limits.

We interpret the data to indicate that fresh formation water containing o-xylene enters the well during the initial hours of recharge after the well is pumped down. The concentration of o-xylene in the water column is then apparently depleted over time due to volatilization losses. The sampling protocol can affect the results obtained from slowly recharging wells relative to volatile organic concentrations.

SUMMARY AND CONCLUSIONS

This paper has presented results of three phases of the investigation of the Wilsonville hazardous waste disposal site: determination of the hydraulic conductivity of materials underlying the site, evaluation of distribution of contaminants, and testing of experimental protocols for sampling of volatile organic compounds. The hydraulic conductivity portion of the study is essentially completed. Results of laboratory tests of hydraulic conductivity were found to be consistently lower than those from in-situ field tests for most materials at this site. This is thought to be due to the fact that laboratory tests do not generally indicate the effects of macrostructures such as joints and sand lenses. In-situ tests using angle holes were used to assess the significance of vertical joints. The in-situ hydraulic conductivity tests and the geologic investigation both revealed that the upper portion of the glacial till underlying the trenches was highly jointed. The differences between predicted and actual migration rates, therefore, can be explained by the fact that only laboratory determined values of hydraulic conductivity were used for the predictions.

Groundwater sampling revealed that the highest degree of contamination was within the unaltered, basal Vandalia Till. The depth of contaminant migration was greater than expected based on measurements of hydraulic gradient and, as such, the pathway is uncertain. The pathway may involve migration through joints and along shear planes, basically controlled by the fracture network. Flow may also have occurred through sand lenses and through the fine-grained matrix.

Time-series sampling of monitoring wells prior to, during and after pumping revealed that o-xylene concentrations reached a maximum after 2 to 8 hours of recharge to the well. Additional studies of the behavior of other compounds, currently underway, are necessary before a protocol can be proposed for sampling of volatile organics in fine-grained, low-permeability materials.

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*Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

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RECLAMATION AND REDEVELOPMENT OF UNCONTROLLED HAZARDOUS WASTE SITES

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ABSTRACT

There are numerous cases in the United States where uncontrolled dumping or industrial spills have contaminated properties with hazardous materials (now more than 18,000 sites inventoried by U.S. EPA). Since many of these properties are in prime urban locations, issues surrounding the reclamation and redevelopment of contaminated properties have assumed national importance. The principal objective of this study has been to document with case studies relationships between site remediation methods, cleanness criteria, and redevelopment land uses.

After extensive interviews with Federal and State officials in all 50 states, 12 uncontrolled hazardous waste sites were selected for detailed study. For each of these sites, remedial actions have been undertaken with some upgraded redevelopment of the property in mind. Redevelopments include single- and multi-family residential, recreational, commercial, institutional, and light industrial land uses.

Two distinctly different types of redevelopment efforts were encountered--public-initiated projects and developer-initiated projects. In the case of public-initiated projects (for example, most Superfund sites), immediate concerns for community health are paramount, and site reuse, if any, tends to be incidental to site cleanup.

In the case of developer-initiated projects, the developer attempts to recover site cleanup costs through resale of the property. Thus, he simply diverts into cleanup operations money that would otherwise be used to purchase uncontaminated land. The economic feasibility of a developer-initiated project may depend directly on the standard of cleanness required of a site for a particular redevelopment type. Since property decontamination standards and guidelines have not been formulated for most situations, some confusion exists and hence, developers generally view contaminated site reclamation/redevelopment projects as risky ventures.

INTRODUCTION

In May 1983, RTI began a study to identify and document specific instances where uncontrolled hazardous waste sites had been cleaned up and redeveloped for some other, upgraded land use. One focus of the study involved careful analysis of the various standards and criteria for cleanup that have been or could be employed in these remediation/redevelopment efforts. It is anticipated that through this ongoing effort a more complete understanding of relationships between site reuse, remediation, and cleanup criteria will emerge to benefit reclamation/reuse efforts.

BACKGROUND AND PURPOSE

Prior to 1976, few states had regulatory programs for land disposal of hazardous wastes. However, national awareness of hazardous waste problems increased dramatically in the mid-to-late 1970's as it became evident that mismanagement of hazardous wastes could lead to the release of toxic materials into the soil, water, and air. Congress responded to the concern over uncontrolled hazardous wastes by enacting the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA), better known as Superfund. Under the CERCLA program, 538 sites with hazardous waste contamination are included as of October 1984 in the Superfund National Priority List (NPL) with an additional 238 sites proposed for inclusion (U.S. EPA, 1984a and b). The extent of the problem is much larger, however. To date, the U.S. Environmental Protection Agency (EPA) has inventoried more than 18,000 uncontrolled hazardous waste sites.

In view of the large number of uncontrolled hazardous waste sites in the U.S. and the effort required to reclaim this land for productive use, issues related to uncontrolled hazardous waste site remediation and redevelopment are of national significance. However, because site remediation and reuse are relatively new public concerns, a great deal of information has not previously been assembled to describe instances where site redevelopment has occurred following site cleanup. This study attempts to make a beginning in this new area.

APPROACH

The project consists of three components: (1) identifying suitable sites for analysis, (2) preparing detailed case studies on selected sites, and (3) determining what criteria exist for guiding the extent of site cleanup that is necessary. Through all three of these study components, the emphasis is on the functional relationships among site remediation methods, cleanup criteria, and redevelopment objectives.

To identify sites that might serve as case studies for remediation and reuse, we conducted telephone and personal interviews with Federal and State officials involved with uncontrolled hazardous waste site cleanups within the U.S. In addition to contacting environmental officials in all 50 States, we have solicited information from each of the ten EPA Regional Offices and each of the ten EPA Regional Superfund Offices. Inquiries have also been made to the U.S. Army Toxic and Hazardous Materials Agency (USATHAMA), the U.S. Army Corps of Engineers, and the Regional Offices of the U.S. Department of Housing and Urban Development. This approach has been highly successful and has led to the identification of sites for detailed study. A carefully designed literature search of 13 online data bases was also conducted, but proved to be of limited value, since there is rarely any mention of site redevelopment in journal and newspaper articles dealing with hazardous waste sites.

In many cases, we were able to obtain copies of pertinent project records, local newspaper articles, and various types of government and contractor reports pertaining to specific sites that were identified through telephone contacts. These materials were reviewed carefully to decide if a particular site was appropriate for further study.

Several sites were identified as candidates for detailed case studies. An effort was then made to assemble complete files on these sites to describe site location, ownership, and special characteristics; land use history and redevelopment objectives; the nature of the contamination; the remediation actions taken or planned; and the specific criteria used to guide the cleanup effort. This detailed information gathering required additional telephone interviews and, in some cases, site visits. Site visits proved to be highly effective in assembling the background documentation for several sites.

We discussed the "how-clean-is-clean?" issue with numerous environmental officials involved in site remediation and reviewed the criteria that were applied for the sites chosen as case studies for the project. In addition to the criteria based on Federal guidelines for air and water, we sought other types of quantitative guidance, particularly standards applicable to soils, that could be applied to site cleanup. These various standards and criteria were compiled to facilitate comparison with the types and levels of contaminants encountered at hazardous waste sites. Information for this work was culled from many sources including various Regional and Program Offices within EPA, the U.S. Army Medical Bioengineering Research and Development Laboratory, the Centers for Disease Control, and the British Department of the Environment.

RESULTS OF THE STUDY

Site Redevelopment Case Studies

During the initial phase of the study, 12 uncontrolled hazardous waste sites located in seven states have been addressed as detailed case studies to document land reuse. Additional sites where reuse is being planned have been identified for further investigation.

Case study sites include former DOD properties, a defunct coal gasification site, abandoned chemical recovery and drum recycling facilities, a former steel mill, munitions and fertilizer manufacturing sites, and several uncontrolled dump sites. Land reuses at these sites include industrial parks, a marina, a recreation park, a hotel and convention complex, single family residences, a public school, residential condominiums, a housing complex for handicapped and elderly, a neighborhood playground, and State offices and facilities. Documentation of these redevelopment case studies has been the major focus of the project. Brief descriptions of the sites that were examined in detail are provided in Table 1.

Six of the land reuse case studies are located in California. The remediation/redevelopment efforts at all the California sites that we investigated were initiated by developers seeking to upgrade the land use. Concerns at these sites pertained mainly to potential exposure of persons who might live or work at the site following redevelopment. In most cases, material and soil that was determined by the California Department of Health Services (CDHS) to be hazardous was removed to a permitted disposal facility. All of the sites that we investigated in California are located near large metropolitan areas.

In Hercules, CA, the former site of the Hercules Powder Works which manufactured dynamite and other munitions from 1912 until 1963, three cases of successful redevelopment were examined. A residential subdivision (single family homes) with a public school was developed as Bayside Village on the southernmost portion of the former Hercules property following a very stringent remediation effort in 1981. Cleanup operations on a second tract of the property were completed in 1983 by Bio-Rad Laboratories, and an industrial park is currently being developed there. Another tract of 50 acres was also cleaned up in 1983 to make way for the residential condominiums known as Hercules Village.

Residential condominiums also have been developed at former uncontrolled hazardous waste sites in Huntington Beach and Yorba Linda, CA. Contamination at these sites stemmed from dumping of refinery wastes including both acid and alkali sludges. The removal operations at these sites were complicated by extensive foul odors from sulfur compounds that were released from the petroleum waste during excavation. In the interest of expediency because of the odor problem, the extent of the removal that was required was left to the discretion of the CDHS representative onsite during the excavation. All waste material and contaminated soil were removed to a landfill permitted to receive hazardous waste.

In South San Francisco, CA, a former steel mill and fabrication plant site has been redeveloped as "The Gateway", a hotel and convention center complex, by Homart Development Company. The remediation agreement stipulated that the location of the contaminated soils be clearly designated on a site map and that these areas not be excavated or substantially disturbed in the future without CDHS approval. A deed restriction was negotiated as a way of enforcing these provisions over time.

The Dade County Transit Authority has plans for a maintenance facility at the former Miami Drum Services site in Miami, Florida. The contamination resulting from the drum recycling operation caused major concern because the Biscayne Aquifer which supplies the drinking water for Dade County lies only one meter below the natural ground surface at the site. The extent of the initial soil excavation was based on the following criteria:

- Soils obviously contaminated as indicated by the total metals analysis were removed. If levels were in excess of ten times "minimum criteria" for groundwater, the soils were generally considered to be contaminated.
- Soils with highly colored, oily deposits as indicated by visual inspection of corings were removed.
- Known locations that received runoff from the drum-washing operations were excavated.

The criterion for removal based on ten times "minimum criteria" refers to guidelines established by the State of Florida for groundwater. These guidelines are consistent with the National Primary Drinking Water Standards for barium, cadmium, chromium, lead, silver, and selenium, but are more stringent for arsenic and mercury. Although total metal concentrations in the soil were used as guidance in the initial excavations, final excavations were guided by the results of chemical tests together with engineering and scientific judgment.

The former Chemical Metals Industries, Inc., in Baltimore, Maryland, also presented an immediate potential hazard. In this case, the major concerns included imminent threat of fire or explosion in the residential neighborhood due to the chemical incompatibilities of the materials present and to the potential posed by runoff from the site.

Remediation and redevelopment at the Kapkowski Road site in Newark, New Jersey, is just getting started. Pockets of PCB-laden oil are being eliminated through a series of oil recovery wells. The site was used from the mid-1950's as an uncontrolled dump site for solid refuse and waste oil. The property, adjacent to the Newark Airport, is a prime location for development as an industrial park. The extent of the remediation effort that will be performed there will be determined when excavation begins for building. During construction if soils are encountered that contain more than 5 ppm of PCB's, the contaminated material will be removed to a permitted disposal facility.

At Plattsburgh, New York, a recreation park now occupies a site where a coal gas generating plant was operated from 1896 until 1960, most recently by the New York State Electric and Gas Corporation (NYSEG). Large quantities of coal tars stored in unlined ponds resulted in the contamination. The site remediation consisted of containment onsite and a cement-bentonite partial cutoff wall to arrest any further migration of the contamination into the Saranac River. In allowing the material to remain onsite, the New York State Department of Environmental Conservation (NYSDEC) has imposed certain restrictions to development. Additional restrictions are also under consideration. These include the following:

- Sale of the lands on which the containment cell was constructed is prohibited by NYSDEC.
- No structures or other activities may be placed or performed on the containment cell that could result in rupture to the Hypalon cap.
- All trees and shrubs will be maintained at a distance from the slurry walls such that their mature drip line will not intersect the slurry wall.
- All construction on or near the cement-bentonite partial cutoff wall and/or groundwater collection system must have prior engineering approval of NYSEG.

Among the U.S. Department of Defense (DOD) sites where remediation and redevelopment have been undertaken is the Frankford Arsenal site located in eastern Philadel- phia on the Delaware River. For more than 150 years, this 110 acre site was associ- ated with Federal munitions research, development and production. When the U.S. Army decided to excess the facility in 1976, USATHAMA assumed responsibility for the site cleanup to satisfy the require- ments of the General Services Adminis- tration prior to sale of the property to private developers. A large portion of the old arsenal has been sold to a development consortium and will probably be used by multiple tenants for light industry. The property closest to the water is intended for use as a regional marina and park to be built by the Pennsylvania State Fish Commission. Projected for completion in 1986, development of this 18-acre facility is expected to cost $3 million.

General Observations

From the record searches and interviews, several findings of a general nature emerged. One observation was simply the small number of Superfund-assisted sites that have become available for redevelopment. Many uncontrolled hazardous waste sites do not pose an immediate threat to public health or welfare as long as they are left undisturbed and thus do not meet the criteria to qualify them as NPL sites. As a result, cleanup must be initiated by the local government or by the private sector if site remediation is to be accomplished. Where remediation costs are high, remediation/reuse will be undertaken by the private sector only in areas where land values are so high that cleanup costs can be recovered through rents or resale.

Across the U.S., several contaminated DOD sites have been remediated and turned back to the private sector during the past few years. Efforts to restore hazardous DOD or former DOD properties contaminated by hazardous wastes, unexploded ordnance, and unsafe or unsightly debris will expand under the Defense Environmental Restoration Program (DERP) authorized by Public Law 98-212. DERP covers both active installations and formerly used DOD properties.

A central issue for the planning of any site redevelopment is the criteria to be used in determining the extent of cleanup that is to be required. Acceptable concentration limits to establish the extent of cleanup that is necessary to protect public health and welfare have not been determined for most toxic substances of concern. A recent GAO report notes that "Superfund provides that long-term remedies be cost-effective, but no standards exist that specify to what extent sites must be cleaned up to effect permanent remedy" (GAO, 1984).

The "how-clean-is-clean?" question is often posed in relation to the National Contingency Plan under CERCLA. Although EPA has received considerable comment regarding the need for allowable levels of release from sites following remediation, the Agency has maintained that the flexibility in the current approach is appropriate because of the site-specific nature of most problems and the need to move ahead in an expeditious fashion.

Almost all States have problems dealing with the "how-clean-is-clean?" issue. Since different uses may imply a need for different cleanup criteria, this type of judgment must be made on a case-by-case basis. Residential use is generally felt to require the most stringent cleanup. Recognized guidelines pertaining to acceptable pollutant levels include air quality standards, occupational exposure guidelines, drinking water standards, and water quality criteria. Although developed for other purposes, these established sets of guidelines are frequently relied upon as criteria for cleanup.

California appears to be ahead of other states with respect to policy and legislation guiding the redevelopment of contaminated land. Within CDHS, a system has evolved during the last 4 years for regulating the cleanup and redevelopment of abandoned industrial sites and waste disposal areas.

Assembly Bill 2370 (AB 2370) is an important feature in the California program for hazardous waste site remediation. This law authorizes CDHS to impose deed restrictions on any tract of land (and surrounding properties) that poses a significant threat to the public health. Though seldom carried to its fullest extent (which requires a public hearing), the existence of AB 2370 gives CDHS much voice in the cleanup of California hazardous waste sites to ensure that adequate cleanup levels are achieved.

California has also developed criteria to explicitly define hazardous wastes. The guidelines contained in the Draft California Assessment Manual (CAM), referred to as the CAM Standards, have been revised several times but are now officially filed, effective October 27, 1984, as formal definitions of "hazardous" and "extremely hazardous" wastes. The CAM Standards were developed to enable a generator to determine if the waste he produces must be managed as a hazardous material. The standards are also applied as criteria to establish the extent of remediation necessary to insure that a site is clean enough to be redeveloped for an upgraded use.

CONCLUSIONS

The first significant finding of our study is that there are many instances in the United States where uncontrolled hazardous waste sites have been or soon will be redeveloped for some upgraded land use. This trend is expected to increase in the future as more sites are remediated. Sites that are redeveloped following hazardous waste cleanups are not easily documented through references to the open literature, for several reasons. First, the brief history of Federally-supported waste site redevelopment efforts in this country has not yet resulted in a long list of completed projects. Second, the substantial costs and investment risks associated with hazardous waste site cleanup operations appear to have discouraged developers from attempting to reclaim many contaminated sites. Third, the delays associated with decisionmaking may stifle redevelopment projects where hazardous wastes are involved. When developers do become involved with an uncontrolled hazardous waste site, they try to perform the necessary remediation to the satisfaction of the regional and local authorities with little publicity if possible.

The uncontrolled waste site development projects that we have found appear to be of two distinctly different types. The first type may be termed the developer-initiated redevelopment effort. Such cases of site cleanup and redevelopment occur in large metropolitan regions and other areas where the locational advantages of a site alone are so great that cleanup costs can be recovered through future resale of the remediated property. In such cases, the decision to remediate and redevelop a specific site is made in the private sector, and the public sector simply regulates and certifies the cleanup process. Such was the case for all of the examples of site redevelopment we identified in California.

The second type of hazardous waste site redevelopment project is the public-initiated project where reuse of the land is clearly secondary in importance to the site remediation that is required for public health and safety. This appears to be the case, for example, with almost all sites on the NPL. For most public-initiated cleanup operations, remediation activities are so complicated and costly that the economic value of the site following cleanup only partially (if at all) justifies the cleanup operation. Where a remediated site passes into public ownership, reuse will probably be determined by the specific property needs of the governing body at that particular location and point in time.

It is important to note that these two types of hazardous waste site redevelopments result from two entirely different motivating forces. In the first case (developer-initiated cleanups), the developer is simply responding to land market forces and diverting into cleanup operations dollars that would otherwise be used to purchase uncontaminated land. In the second case (public-initiated cleanups), the redevelopment decision is made in the public sector, and there is no explicit requirement that cleanup costs be recovered through future use of the property.

Most of the redeveloped sites that we have identified are of the developer-initiated type. Only two sites in reuse were identified that involved Superfund. The complexity of the legal process in dealing with site cleanup is not conducive to deliberate redevelopment efforts. Even in cases where emergency remedial response actions have been completed, it appears that the site may remain in receivership and go unused for long periods of time (typically several years) while the courts decide cost recovery and/or property ownership issues.

California appears to lead other States in the formulation and enactment of legislation and regulations pertaining to the cleanup and redevelopment of properties contaminated with hazardous waste. With adoption of the CAM Standards, California has begun to define quantitatively what is meant by "hazardous waste contamination." Their program for guiding redevelopment of contaminated land appears to be the most advanced State program in the Nation.

Based on our investigation to date, we believe that planning efforts involving uncontrolled hazardous waste site mitigation and reuse can benefit from the learning experiences of others involved in similar activities. Therefore, an ongoing effort to assemble the type of information provided through this study could provide a valuable source of information for Federal, State, and local authorities. Such information would also be of great value to developers in the private sector who, having more knowledge of actual procedures required to decontaminate properties, might be persuaded to get involved in such activities.

ACKNOWLEDGMENTS

The authors wish to acknowledge RTI Economist, Mr. Richard Harper, for his assistance in information gathering during the initial phase of the project. We also acknowledge EPA staff members--Project Officer, Ms. Norma Lewis; Mr. Don Sanning, Program Manager for Remedial Action Investigation; and Ms. Naomi Barkley, who has recently replaced Ms. Lewis as the official Project Officer--for their guidance and helpful comments throughout the study. We are very grateful for the cooperation shown by the various local, State, and Federal officials who have

answered our questions and provided information on specific sites.

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TABLE 1. REDEVELOPMENT SITES SELECTED FOR CASE STUDIES[a]

Site and Reuse Description	Nature of Contamination	Remediation
Bio-Rad Laboratories 175 acres Hercules, CA Reuse: Industrial Park	Formerly used for TNT production by Hercules Powder Works; numerous deteriorating metal drums and soil contaminated with heavy metals (Pb, Zn, Cu, Cd) and explosives.	Drums removed; contaminated soils excavated. More than 1,835 cu m (>2,400 cu yd) of soil and debris removed to a permitted hazardous waste disposal facility.
Hercules Village 50 acres developed by D & S Company Hercules, CA Reuse: Residential condominiums	Formerly used as wastewater treatment area by Hercules Powder Works; contaminated with As, Pb, Cr, Zn, DNT, and DNB.	Almost 7,646 cu m (10,000 cu yd) of soil removed to a permitted hazardous waste disposal facility; some residues of DNT and DNB left onsite and covered by more than 3 m (10-12 ft) of clean fill in area restricted from residential development; 2-year monitoring program.
Bayside Village 100 acres Developed by Citation Builders Hercules, CA Reuse: Single-family residences and public school	Formerly Hercules Powder Works property. Surface and subsurface soil contained Pb at levels as high as 530 ppm.	All contaminated soil removed.
Bolsa Chica 12.5 acres Mola Development Corporation Huntington Beach, CA Cost of Cleanup: >$5 million Reuse: Residential Condominiums (224 units)	Site of uncontrolled dumping of refinery sludges (>1,529 cu m) prior to 1963; permitted Class III landfill from 1963. Contaminated with acidic petroleum wastes (aliphatic and aromatic hydrocarbons, amines, thiophenes); methane and odor.	All 45,873 cu m (60,000 cu yd) contaminated material removed to a permitted hazardous waste disposal facility.
The Gateway 117 acres Developed by Homart Development Company South San Francisco, CA Reuse: Hotel and Convention Center	Formerly a steel mill and fabrication plant using coal-fired open hearth furnace. Soils contaminated by heavy metals, oils, and acids deposited over 75 years. Localized high concentrations of As, Cr, Zn, Cu, Ni, low pH, and PCB's.	Due to large volume, majority of contaminated materials contained onsite and capped. More than 700 cu m of hazardous material removed to offsite disposal facility.

TABLE 1. (Continued)

Site and Reuse Description	Nature of Contamination	Remediation
Kellog Terrace 21 acres Gfeller Development Company Yorba Linda, CA Reuse: Residential Condominiums (224 units)	Site of sand and gravel excavation prior to 1940; uncontrolled dumping of refinery waste and used drilling muds in 1940's. Contaminated from Pb, As, and aliphatic and aromatic hydrocarbons.	Approximately 7,034 cu m (9,200 cu yd) of contaminated soil and fill removed to a permitted hazardous waste disposal facility.
Annapolis Road Sites Baltimore, MD Cost of Cleanup: >$325,000 Reuse: Maryland Department of Health Office; Neighborhood Park	Sites used by Chemical Metals Industries, Inc. for recovery of precious metals from waste chemical solutions and printed circuit boards. Large number of leaking drums (corrosive liquids, ammonia- or cyanide-bearing materials, organic solvents); storage tanks, unstable powdered and solid zirconium metal; high levels of Cd in soil. Contaminated groundwater.	More than 1,500 plastic and metal drums removed. Waste oil, water and gasoline pumped from underground tanks. Walls decontaminated by sandblasting. Approximately 91 metric tons (100 tons) of contaminated soil and debris and 175 drums of solids and sludges removed to a permitted hazardous waste disposal facility. Aboveground tanks emptied and removed. Zirconium metal removed.
Miami Drum Site 1 acre Miami, FL Cost of Cleanup: $1.6 million Reuse: Dade County Transit Authority Maintenance Facility	Used for 15 years by Miami Drum Services for drum recycling; 400 to 500 used drums left onsite. Surface and subsurface soils and groundwater contaminated from spills--phenols, heavy metals, oil and grease, and pesticides.	Hazardous debris and contaminated soil (7,335 cu m) removed to permitted hazardous waste disposal facility. Marginally contaminated soils left in place. Groundwater treated onsite.
Kapkowski Road Site West Area--134 acres East Area--111 acres Elizabeth, NJ Reuse: Elizabeth Industrial Park by the Port Authority of New York and New Jersey	Uncontrolled dump site from mid-1950's to 1972. Contamination from waste oil containing PCBs up to 4,800 ppm. Between 1 and 3 million gallons of oil floating on water table and adhering to materials in refuse, primarily in the west area.	Pump out free floating oil from pockets; leave refuse in place. Soils with PCB's greater than 5 ppm to be disposed to a permitted facility if such soil is discovered during excavation for building foundations.

TABLE 1. (Continued)

Site and Reuse Description	Nature of Contamination	Remediation
New York State Electric and Gas Corporation (NYSEG) site 11 acres Plattsburgh, NY Reuse: Recreation Park on the Saranac River	Formerly a coal gasification plant operated by NYSEG. Soil and sediment from the Saranac River contaminated with coal tar leached from unlined coal tar ponds.	Subsurface migration of tar from original disposal ponds arrested by soil-bentonite slurry wall. Contaminated sediment from river bed excavated and removed to containment area surrounded by slurry wall. Containment area capped with Hypalon liner. Cement-bentonite cutoff wall constructed to prevent migration of uncontained coal tar into the river.
Frankford Arsenal Site 110 acres to be developed by Shetland Properties; 18 acres to be developed by Pennsylvania State Fish Commission Philadephia, PA Cost of cleanup: $8 million Reuse: Regional Park and Marina; Light Industrial Development	Formerly used by the U.S. Army for munitions manufacture and research development of armaments. Low-level radiological contamination in buildings, deposits of explosive/pyrotechnic residues, unexploded ordnance, inorganic chemical residues in buildings and underground waste discharge system.	Explosives residues destroyed. Radiological and heavy metal contamination removed from the site.
The Courtyard Developed by Vermont Associates Winooski, VT Cost of Cleanup: $15,000 Reuse: Housing Project for the Handicapped and Elderly	Warehouse used for storage of industrial chemicals; formerly occupied by a silk-screening firm. Approximately 5.66 cu m (200 cu ft) of solid chemical wastes beneath wood flooring; traces of organic solvents, oil.	Underground tanks emptied; 84 metric tons (93 tons) of solid inorganic waste and contaminated wood flooring removed to a permitted hazardous waste disposal facility. Vapor barrier installed and covered by 4 inches of concrete.

[a] Information summarized from case studies in Ray and Kingsbury (1984).

REMEDIAL ACTION MODELING OF THE WESTERN PROCESSING AND OCCIDENTAL-LATHROP SITES

by
David B. Watson[1]
Fredrick W. Bond[2]
Stuart M. Brown[3]

ABSTRACT

Data from the Western Processing and Occidental-Lathrop hazardous waste sites were used to demonstrate, and to the extent possible, confirm, the utilization of numerical computer models for the evaluation of remedial actions at hazardous waste sites. Ground-water flow and contaminant transport models of the sites were developed and calibrated against existing data. The Western Processing model was used to assess and rank possible remedial actions for the site.

The Occidental-Lathrop site model was used to evaluate the effectiveness of an installed ground-water extraction, treatment, and reinjection system. Model results helped to confirm the use and limitations of numerical models for the evaluation of hazardous waste sites.

INTRODUCTION

Proper design and operation of remedial action systems at hazardous waste sites is crucial in terms of human health/environmental risk reduction, costs, design life, and probability/consequences of failure. Assessment of remedial action alternatives and evaluation of the effectiveness of installed systems are important to insure proper design and efficient operation. Techniques for evaluation of remedial action systems include desk-top procedures for screening remedial action alternatives, and complex, numerical computer models for detailed site evaluation. Detailed evaluation of two sites has been conducted with numerical computer models: 1) the Western Processing (WP) hazardous waste site located south of Seattle, Washington, within the city limit of Kent (Figure 1); and 2) the Occidental Chemical Company (OCC) hazardous waste site located south of Lathrop, California, in the Central Valley (Figure 2).

The WP facility was in operation from 1957 through 1982, and consisted of a solvent recycling plant, a fertilizer plant, bulk storage tanks, drum storage areas, and large above-ground storage lagoons for liquid wastes, cooling water, and process water (EPA, 1983). Inspections made in May 1982 by the EPA and Washington State Department of Ecology (WDOE) indicated that drums and bulk storage tanks were in poor condition and were leaking, and numerous areas of spilled material were observed. Twenty-six of the priority pollutants were found in soil and ground-water samples taken from the site. As a result of these findings, operations at the site ceased in 1982. Initial mitigative measures were implemented (limited drum inventory and storage, limited excavation and removal, etc.) and studies were initiated to determine a permanent remedial action for the site.

From 1953 until the present, the Lathrop plant has produced a wide range of agricultural products, primarily pesticides and fertilizers. Historically, liquid process wastes were disposed of in several unlined surface impoundments and ditches, and solid pesticide wastes were buried in an area known as the boneyard.

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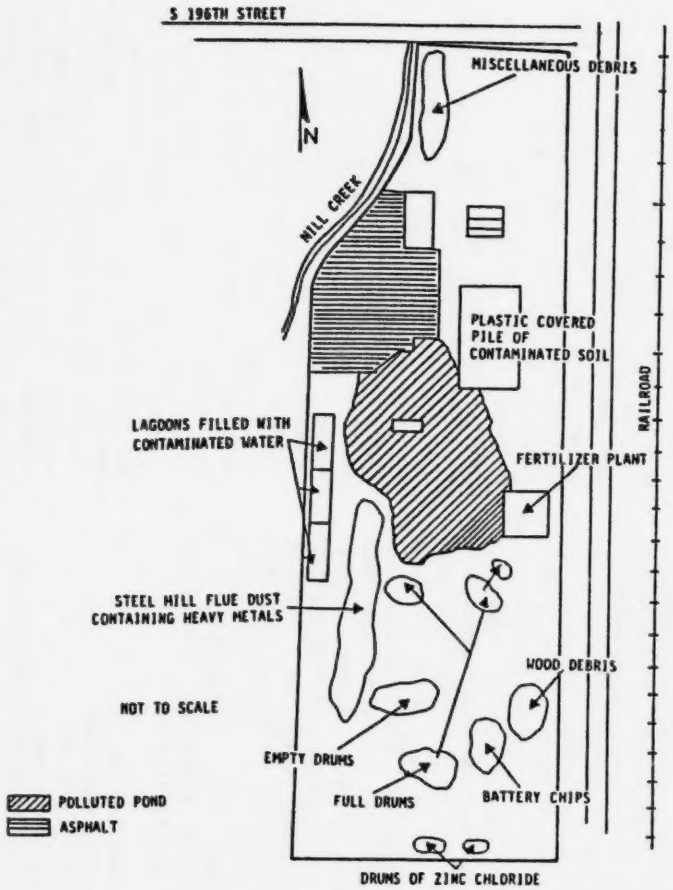
S 196TH STREET
MISCELLANEOUS DEBRIS
N
MILL CREEK
PLASTIC COVERED
PILE OF
CONTAMINATED SOIL
RAILROAD
LAGOONS FILLED WITH
CONTAMINATED WATER
FERTILIZER PLANT
STEEL MILL FLUE DUST
CONTAINING HEAVY METALS
WOOD DEBRIS
NOT TO SCALE
EMPTY DRUMS
FULL DRUMS
BATTERY CHIPS
POLLUTED POND
ASPHALT
DRUMS OF ZINC CHLORIDE

Figure 1. Western Processing Site Map.

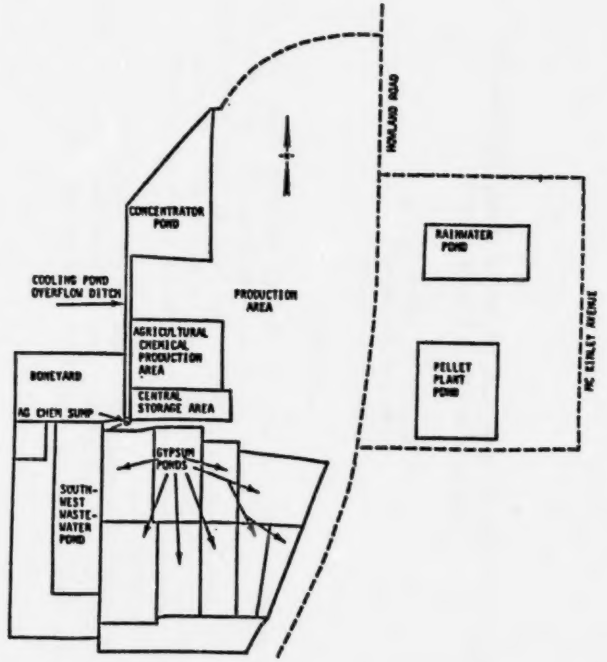
HOWLAND ROAD
CONCENTRATOR POND
RAINWATER POND
COOLING POND OVERFLOW DITCH
PRODUCTION AREA
MC KINLEY AVENUE
AGRICULTURAL CHEMICAL PRODUCTION AREA
BONEYARD
PELLET PLANT POND
CENTRAL STORAGE AREA
AG CHEM SUMP
GYPSUM PONDS
SOUTH-WEST WASTE-WATER POND

Figure 2. Occidental Chemical Company, Lathrop, Site Map (adapted from Canonie, 1981).

The possibility of ground-water contamination was first brought to the attention of the California Regional Water Quality Control Board (WQCB) in 1978. In March 1979 the WQCB issued a cease and desist order directing an immediate end to the discharges on the site and compliance with a site assessment and clean-up schedule because of the public health threat from 1, 2 dibromochloropropane (DBCP) contaminated ground water (U.S. EPA, 1984). At this time, both DBCP and sulfate had been found in private wells used for drinking water. Subsequently, excavation and clean-up of the site began under the review of the EPA National Enforcement Investigation Center (NEIC) and the WQCB. Buried pesticide wastes and contaminated soil were removed and a clay cap installed. Fifty-four monitoring wells and an extraction, treatment, and re-injection system were installed for the long-term monitoring and remedial clean-up of the aquifer beneath the OCC site. The remedial system consists of: 1) five wells extracting water from the upper aquifer; 2) a reverse pulse granular activated carbon system to treat the contaminated extract; and 3) two wells injecting treated water into a lower briny aquifer.

PURPOSE

The first objective of this study was to demonstrate, and to the extent possible, confirm, the use of models for remedial action evaluation. The second objective was to provide technical information on the performance of remedial action technologies at actual sites that will improve the evaluation and decision process at those sites. The objective of the Western Processing study was to provide technical assistance to EPA Region X in designing and evaluating remedial action alternatives for the uncontrolled hazardous waste site. The primary objective of the OCC study was to demonstrate the use of numerical computer models for the evaluation of the effectiveness of an active remedial action system. A secondary objective was to provide technical information to the WQCB and California Department of Health Services.

APPROACH

Conceptual models were developed for the WP and OCC sites based on the available hydrogeologic data. The conceptual models formed the framework for developing the flow and transport numerical models. Next, a finite element grid was developed and the appropriate data on stratigraphy, boundary and initial conditions, hydraulic conductivities, and hydraulic stress were input to the code. The Finite Element Three-Dimensional Groundwater (FE3DGW) code (Gupta et al., 1979) was used to simulate ground-water flow in a localized area around both sites.

Contaminant transport was simulated using the Three-dimensional Coupled Fluid, Energy, and Solute Transport (CFEST) code (Gupta et al., 1982). CFEST is an extension of FE3DGW in that it uses the same hydrologic flow data structure and finite element grid. In addition, CFEST includes the necessary parameters to couple contaminant transport with ground-water flow.

WP Site

The top 100 ft of the water table aquifer was modeled at Western Processing. This section was further divided into four horizontal layers to allow for a more detailed description of horizontal and vertical hydraulic conductivity.

The difference between predicted and measured hydraulic potentials was minimized in the flow calibration process by adjusting vertical and horizontal hydraulic conductivities, flow to a creek and drainage ditch within the study area, and boundary conditions.

Trichloroethylene (TCE) was chosen for simulation in the transport model because TCE has been measured in high concentrations, and it is one of the most ubiquitous contaminants at the site. The difference between predicted and measured TCE concentrations was minimized in the calibration process by adjusting the retardation factor, infiltration rates, and source strengths.

Once the model was calibrated, it was used to predict the effectiveness of six proposed remedial action alternatives: 1) no-action; 2) source removal; 3) cap; 4) source removal combined with a cap; 5) upgradient slurry wall combined with a cap; and 6) pump and treat. For each of the remedial actions simulated, the flow portion of the model predicted changes in the flow field and volumes of water removed, while the transport portion of the model predicted the mass of contaminant removed and average concentrations up to 25 years into the future.

OCC Site

The aquifer at the OCC site consists of 250 feet of interbedded clay, silt, sand and gravel (CDM, 1981). Upper, middle, and lower aquifer zones were simulated, and five material types were defined based on the areal distribution of percent sand and gravel reported in well driller's logs.

The flow model was calibrated by minimizing the difference between predicted and measured potentials through the adjustment of material properties, boundary conditions, and the quantity and location of agricultural pumpage.

Because of its toxicity, mobility and persistence, DCBP has been the target chemical of the remedial action at the OCC site. Initial plume distribution for the the transport model was estimated from measurements of DBCP concentrations in samples taken from the permanent monitoring wells during October 1982.

The transport model was then used to: 1) compare measured and predicted plume concentrations; 2) compare measured and predicted DBCP concentrations at the remedial extraction wells; and 3) predict long-term effectiveness of the remedial system.

PROBLEMS ENCOUNTERED

At both sites the flow model calibration was hindered by a lack of hydrologic data. Extensive hydrogeologic data exists for the 13-acre Western Processing Site, but very little data exists for the area surrounding the site, and for depths greater than 30 feet below the surface. At the OCC, site agricultural pumpage to the west and south of the site was the most important variable in the flow calibration process. However, pumping schedules at these wells were not available. Estimation of recharge at two waste water discharge ponds and estimates of infiltration rates due to irrigation were also problems for the OCC site.

At both sites there was a lack of data regarding transport parameters and operation history. Little or no detailed data exists regarding when and where contamination occurred, and the quantity and types of chemicals present. For Western Processing, no site-specific data were available regarding partitioning coefficients and dispersivities. This lack of data resulted in assumptions regarding contaminant transport. Temporal and spatial variability in DBCP concentrations, measured at the monitoring wells, made detailed mapping of the plume difficult for the OCC site.

RESULTS

WP Site

A ground-water flow and contaminant transport model of the Western Processing Site was developed and calibrated. The calibration effort was based on comparison of predicted and observed hydraulic potentials and TCE concentration in the creek within the study area.

The model results showed that the creek has been and will continue to be the primary discharge point for TCE. By 1983, almost half the TCE estimated to have entered the flow system during site operation had exited to the creek. Over the next 25 years, the model predicted, for the no-action case, that about 60% of the TCE remaining in the system in 1983 will have exited to the creek.

The results of the remedial action simulations are summarized in Figure 3. Of the remedial actions simulated, pump and treat removed the most TCE from the flow system. Of the passive actions, source removal with or without a cap was the most effective. None of the simulated actions prevented TCE from discharging to the creek, but the pump and treat and source removal actions were most effective in reducing the amount. The capping and slurry wall options simulated were ineffective in terms of significantly reducing the movement of TCE to the creek.

OCC Site

The calibration of the flow model was based on comparison of predicted and measured hydraulic potentials. The match between predicted and measured hydraulic potentials was very good for the October 1980 to September 1982 period. For the October 1982 to February 1984 period, the comparison was not as good as the earlier period, however, it was determined further calibration of the flow model would have little affect on the transport model results.

October 1983 DBCP concentrations were generally under-predicted in the center of the plume by a factor of approximately

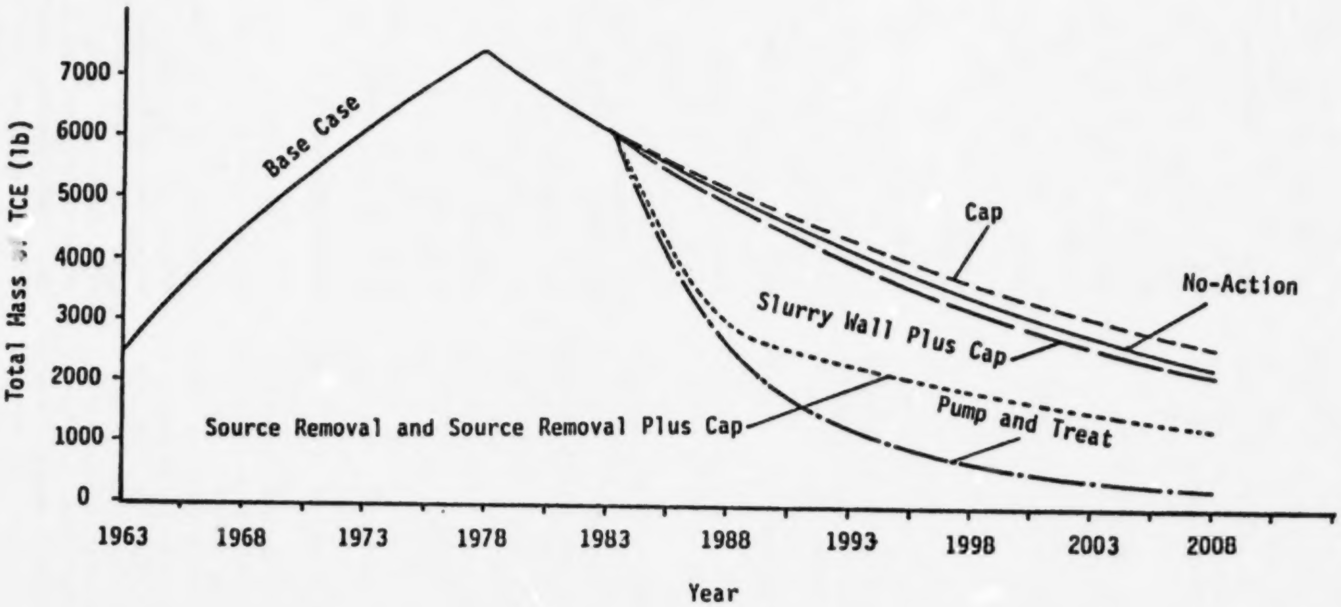

Figure 3. Comparison of the Total Mass of TCE Remaining in the Ground-Water System for the Five Remedial Action Cases.

1.5. Concentrations on the fringe of the plume (location of 10 ug/l contour) were predicted with reasonable accuracy. A comparison of predicted and measured percent plume reduction for October 1983 in the shallow, middle and deep zones is presented in Table 1. On the whole, plume reduction was under-predicted, especially in the middle zone where most of the remedial extraction wells are screened.

Table 1. MEASURED AND PREDICTED DBCP PLUME REDUCTION

Zone	Measured	Predicted
Shallow	20%	5%
Middle	50%	15%
Deep	20%	20%

Composite influent concentrations to the treatment plant were greatly under-predicted at early times and slightly over-predicted at latter times. Figure 4 shows the computed and observed concentrations at each extraction well. Concentrations are slightly over-predicted for EW-1, 2, and 4 and greatly under-predicted for EW-3 and 5.

The difference between predicted and measured results (presented in Table 1 and Figure 4) are probably caused by heterogeneities such as sand and gravel lenses that are not represented by the transport model and not completely quantified by the site data, and by spatial variability in the plume. DBCP is probably concentrated and transported in preferred pathways such as sand and gravel lenses or layers. An extraction well near one of these preferred pathways will probably withdraw much of its water from this sand and gravel layer, especially at the beginning of withdrawal. Therefore, concentrations measured in these wells should initially be greater than concentrations measured at the unpumped monitoring wells. Reports that extraction well concentrations were much greater than expected during start-up of the remedial system supports this contention (Roy F. Weston Consultants, 1984). Because the plume distribution utilized in the model was estimated from monitoring well data, the transport model under-predicted DBCP concentrations at several of the extraction wells in contact with a preferred pathway (EW-3, and 5). Spatial variability in the plume made it impossible to estimate the concentration of DBCP at any individual extraction well. Plume reduction was under-predicted because the DBCP is concentrated in the preferred pathways and not spread throughout the entire aquifer as modeled.

Long-term model predictions show substantial DBCP remaining in the aquifer after ten and twenty years. Stagnation of the plume occurs after ten years suggesting that the extraction wells may have to be moved for complete capture of the plume. Significant movement of the plume off-site was not predicted. However, irrigation wells south and west of the site were predicted to capture some of the plume. Long-term model results may change if the model is altered to represent transport of DBCP through preferred pathways.

CONCLUSIONS

Results from the two modeling studies show that numerical computer models are helpful for the evaluation of remedial actions at uncontrolled hazardous waste sites and analysis of the effectiveness of installed remedial systems, but they have limited ability to accurately quantify contaminant transport especially at complex sites with many unknowns concerning the temporal and spatial attributes of transport. However, the modeling of both sites helped to identify data deficiencies and improve the understanding of contaminant transport at the sites. Modeling of the WP site facilitated the ranking of remedial action alternatives. Results from the OCC study shows the importance of re-evaluating the conceptual model and recalibrating the numerical model as more data becomes available.

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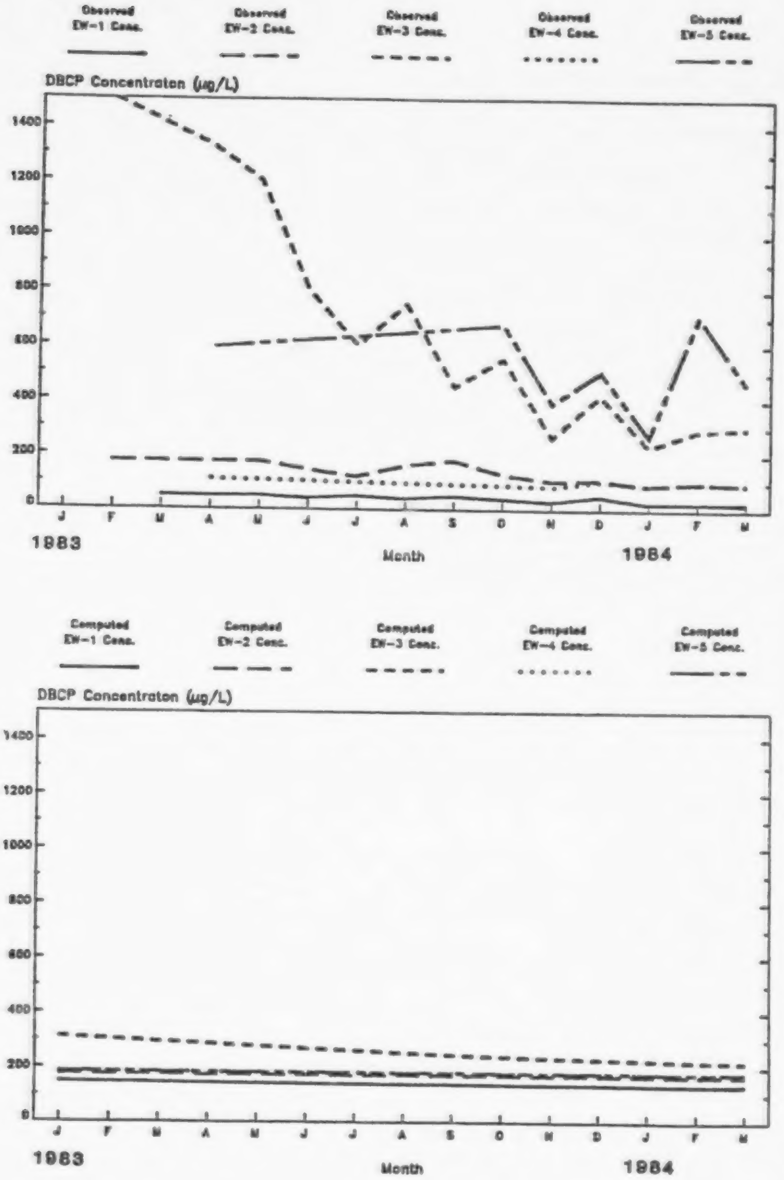

Figure 4. Comparison of Computed and Observed Extraction Well Concentrations at OCC Site (adapted from Roy F. Weston Associates, 1984).

Data for the Occidental Chemical Company Study were made available from Mr. Howard Hatayama, California Department of Health Services and Mr. Thomas Pinkos, Water Quality Control Board.

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DEMONSTRATION OF A GEOGRAPHIC INFORMATION SYSTEM FOR HAZARDOUS WASTE SITE ANALYSES

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ABSTRACT

This paper investigates the applicability of a Geographic Information System (GIS) to the study of a hazardous waste site. Existing data, including aerial photographs, maps, and site study reports, were digitized to create a spatial data base for an active Superfund site. These "computer maps" were then analyzed by cartographic modeling techniques to assess current site conditions and prospective remedial actions. Existing site drainage basins were identified and characterized by whether stormwater ran off-site or infiltrated on-site. Wastes were inventoried within each basin and stormwater/waste-budgets were developed for each drainage basin. Additional "computer maps" were then constructed to represent remedial action scenarios. Stormwater/waste-contact budgets were developed for various combinations of berms, dikes, and asphalt covers. These alternatives were evaluated according to the estimated reductions in waste contacted runoff and infiltration. Excavation volume estimates were developed for the above-grade waste types and successive estimates were accomplished for removing contaminated soils down to the groundwater table.

The application of a GIS to hazardous waste site investigations offers significant advantages. In addition to the novel analyses demonstrated in this report, a GIS can provide for more efficient data storage, availability, and utilization than currently practiced in most site investigations. Several improvements in EPA data management capabilities and practices, however, will be required. Specific recommendations are offered. Phase II of this project involves a workshop instructing EPA personnel and associates in the use of a GIS at a hazardous waste site.

INTRODUCTION

There has been a tremendous increase in both the volume of information collected and the types of studies and analyses conducted at waste sites regulated under the Resource Conservation and Recovery Act (RCRA) and the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA or "Superfund"). The procedures employed at both RCRA and Superfund sites often require several years and involve many agencies and contractors collecting and analyzing data for different purposes. Unfortunately, as these projects move forward, there may be abrupt changes in the personnel and the level of attention addressed to a site. New investigators may be unaware of unpublished data sources collected earlier in a project. Many studies have a short-term focus and the data collected are inappropriate or do not meet quality control standards. Much information is lost because few data management systems are flexible enough to accept, inventory and format particular types of data (such as maps or aerial photographs) collected at hazardous waste sites. These problems result in inefficient use of resources and often expensive duplicative efforts to restore or recollect lost or inadequate information.

The use of Geographic Information Systems (GIS) can provide a framework for resolving several of these difficulties. A GIS is a computer-based system capable of assimilating and inventorying spatially-oriented data from a number of sources in a single location-specific or "computer map". It is also capable of manipulating those data in a variety of processes called cartographic modeling. Data from several sources can be concurrently analyzed without redevelop ing information collected in earlier phases of a project. The system can provide for the storage of almost all types of available data, facilitate the evaluation of data from outside sources, and aid in the design of study plans that consider the long term value of any information collected.

This paper reports on Phase I of a two-phase project to investigate the applicability of a GIS in the management and analysis of a hazardous waste site. In Phase I a spatial data base was developed for an existing hazardous waste site. Cartographic modeling and geostatistical analyses techniques were applied to the data base to demonstrate the flexibility and capabilities of a GIS.

In Phase II of the project, a GIS workshop for EPA project personnel and associates will be developed. A lecture/exercise format will be used to apply the GIS to specific hazardous waste site problems, including those demonstrated in Phase I. EPA's National Computer Center (NCC) computer at Research Triangle Park, North Carolina will be utilized. The three-day workshop will follow this Research Symposium in Cincinnati, Ohio in May 1985.

PURPOSE

The purpose of Phase I of this project was to demonstrate the applicability of a Geographic Information System (GIS) to a hazardous waste site. Specific tasks were:

1. Select a demonstration GIS suitable for analysis and instruction purposes
2. Select a demonstration site representative of the problems and practices encountered at both controlled and uncontrolled hazardous waste sites
3. Create a spatial data base for the demonstration site
4. Perform analyses to assess current site conditions and prospective remedial actions

APPROACH

Selection of Geographic Information System

The Map Analysis Package ("MAP"), a grid-based GIS available from the Yale School of Forestry and Environmental Sciences, was selected for use in this project (1). "MAP" is an academically oriented GIS developed primarily for educational and instructional purposes. It is an English-based "user friendly" system that employs most of the cartographic analysis techniques used in environmental and natural resource research. It is available in the public domain and is advanced in its analytical capabilities in comparison to many commercial GIS'.

The "MAP" grid cell structure resembles a matrix, consisting of a specific number of rows and columns. Each row-column element or "cell" contains a value representing some dimension or an attribute of that location. It is this value assigned to a cell that allows algebraic manipulation of computer maps. The manipulation of a cell's value in one map, based on either the value of its neighbors or on that same cell's attribute in another map, is called "map algebra" (2). Map algebra is analagous to traditional algebra except individual maps are treated as the independent and dependent variables.

Figures 1 and 2 show how maps may be combined using traditional algebraic operators to produce new maps. In Figure 1, the "point characteristics" of the maps are utilized. The value of a particular cell in one map is combined with the corresponding values in the same cell of other maps to produce a new map.

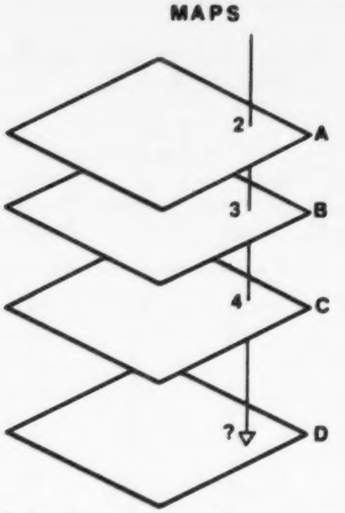


FIGURE 1 A new map, D, is produced by simultaneously considering values of existing maps, A, B, & C, on a cell-by-cell basis.

Figure 2 represents how "neighborhood characteristics" are utilized. Using Boolean algebra in a procedure analagous to traditional map overlay or successsive subtraction techniques, the "neighborhood" of a specific sample location may be characterized. Logical combinations of intersections and unions are utilized.

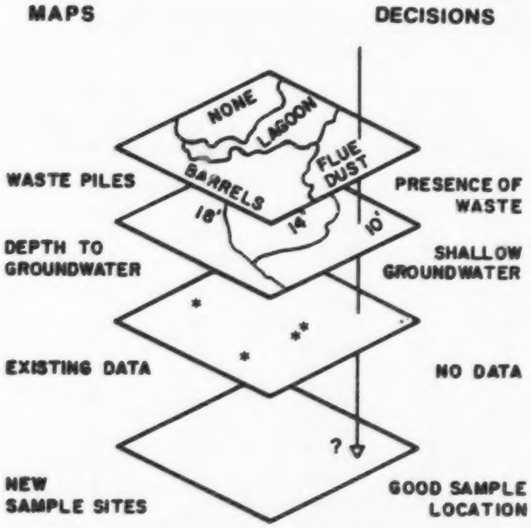


FIGURE 2 A new map, "New Sample Sites", is created by considering the neighborhood of a cell on existing map overlays.

These figures demonstrate the concept of map manipulation. The actual techniques used for the manipulation vary from simple addition to movement across friction or weighted surfaces. Detailed descriptions of the techniques may be found in Tomlin (1).

Selection of Demonstration Site

Western Processing, a Superfund site in Kent, Washington, was selected as a demonstration site representative of the problems, management practices, and data quality encountered at both controlled and uncontrolled hazardous waste projects. The 13-acre site was operated as an Army 70-mm battery and as a privately owned waste recycling and storage facility until 1982. A variety of hazardous wastes had been stored on the site. EPA conducted a detailed site investigation in 1982 during which over 30,000 data points from 30 monitoring wells, 50 soil sample sites and 28 air sample sites were collected. Eighty-seven priority pollutants, including 21 known and 28 suspected carcinogens were found on or near the site (3).

The EPA and the State of Washington began remedial actions in 1983 utilizing berms and impervious surfaces to prevent imminent spillage of materials and to reduce runoff of contact stormwater. The EPA has supported several research investigations of alternate cleanup strategies at this site. They include building decontamination, drum encapsulation, control material weathering, groundwater control and revegetation studies.

Creation of Spatial Data Base

Aerial photographs, maps, slides, reports and data summaries obtained from the EPA Region X Office, EPA Laboratories and contractors were digitized to produce base maps of the site (4). A scale of 225 ft^2 per grid cell was used for each map. The maps, which were 58 columns wide and 116 rows long, and contained 6728 cells equivalent to 34.8 acres, are demonstrated in Figures 3 through 5. A perimeter around the 13-acre site allowed for consideration of surrounding off-site conditions.

The base maps were combined and manipulated to create new or "derived" maps which became part of the spatial data base for the site. For example, an overlay of the PROPERTY LINES, CREEK, BUILDINGS and SITE ROADS base maps yields a new map of the SITE FEATURES. Maps resulting from the manipulation of other maps are distinguished from base maps in that they require no additional data input or digitization but instead are derived from information already contained in the data base.

Demonstration Analyses

Three demonstration analyses utilizing the completed data base are discussed. These analyses address on-site surface characteristics, however these techniques could be applied to off-site groundwater and air pollution problems as well. A brief description of each follows:

1. Identification of Original Site Characteristics

Cartographic dispersion techniques were applied to the original site topography map to identify twenty-seven unique drainage basins for surface runoff waters. Figure 3 illustrates the results of these analyses.

Basins were categorized by whether they drained off-site or ponded and infiltrated on-site. An overall stormwater budget was developed and the basins were ranked by their contribution to site runoff or site infiltration as shown in Tables 1 and 2.

Cartographic inventory methods were then used to characterize the type and volume of wastes in each basin. These results were combined to identify those basins where major stormwater/waste contact occurred and how and where those waters eventually left the site. Figure 4 shows the wastes within the major basins. Resulting inventories for the major basins are summarized in Table 3.

TABLE 1. DESCRIPTION OF SITE BASINS PONDING ON-SITE

Rank	Basin Number (Fig.3)	Area Drained (acres)	% of Total Site Drainage Area	% of On-site Ponding
1	16	2.0	13	24
2	23	1.0	6	12
3	24	.7	4	8
4	20	.6	4	7
4	21	.6	4	7
5	15	.5	3	6
5	18	.5	3	6
6	11	.4	3	5
6	12	.4	3	5
6	14	.4	3	5
6	17	.4	3	5
7	10	.3	2	4
8	22	.1	<1	3
9	9	.2	1	2

53% OF TOTAL SITE RUNOFF
PONDS ON-SITE

TABLE 2. DESCRIPTION OF SITE BASINS DRAINING OFF-SITE

Rank	Basin Number (Fig.3)	Ultimate Destination	Area Drained (acres)	% of Off-site Runoff
1	1	Creek	1.6	22
2	19	West	1.1	15
3	7	Site Ditch	1.0	14
4	3	East	.8	11
4	6	Site Ditch	.8	11
5	8	West	.5	7
6	25	South	.4	5
7	2	North	.3	4
7	7	North	.3	4
7	27	South	.3	4
8	26	South	.2	3
9	5	Site Ditch	<.1	1

47% OF TOTAL SITE RUNOFF
DRAINS OFFSITE

TABLE 3. FEATURES IDENTIFIED WITHIN MAJOR DRAINAGE BASINS

	Basin #	Wastes	No. Cells Containing Waste (1 cell=225 ft^2)
ONSITE	16	Flue Dust	5
		Barrels	21
		Plating Tanks	1
		Storage Tanks	13
	23	Barrels	17
		Lead & Wood Chips	32
		Debris	17
	24	Barrels	56
		Debris	6
OFFSITE	1	Fuel Tanks	1
	19	Flue Dust	44
		Barrels	5
	7	Gypsum Pile	47

This application illustrates the quick use of the initial data base in characterizing site impacts. Both specialized cartographic dispersion modeling techniques and procedures mimicing traditional map reading (overlay and inventory) were utilized. These types of results can be used in the design of sampling strategies, preparing contractor directives, or in developing and evaluating remedial action proposals.

2. Evaluation of Stormwater Control Remedial Alternatives

This analysis also involves stormwater budgets, but illustrates repetitive cartographic modeling techniques. The input topography map was modified to reflect proposed remedial actions. Berms were "built" on the topography map to stop site runoff and the new ponds and infiltration basins were identified and quantified. Areas were then "paved" to prevent infiltration and route uncontaminated stormwater offsite. Several combinations of berming and ponding were evaluated according to their effect on the site's stormwater budget. Figure 5 illustrates one cartographic "construction" of remedial activities. Figure 6 compares the final stormwater disposition of all alternatives considered.

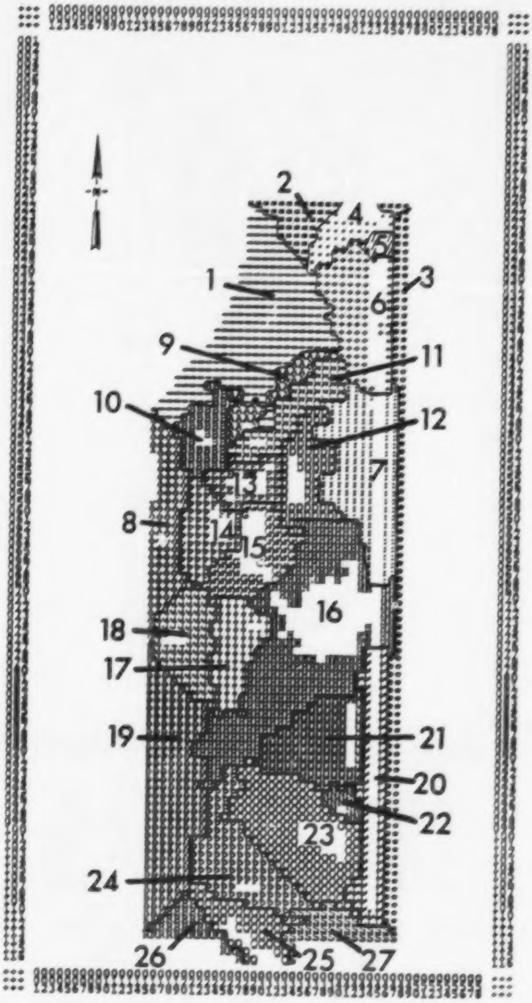
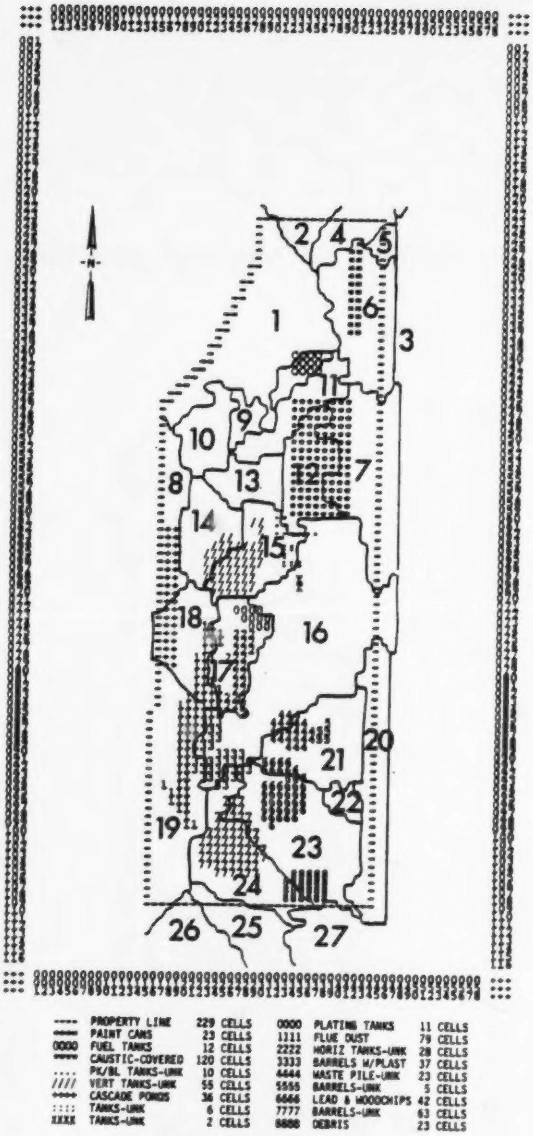


FIGURE 3 Drainage basins and ponded areas (in white) are identified to characterize the Western Processing site status in July 1983.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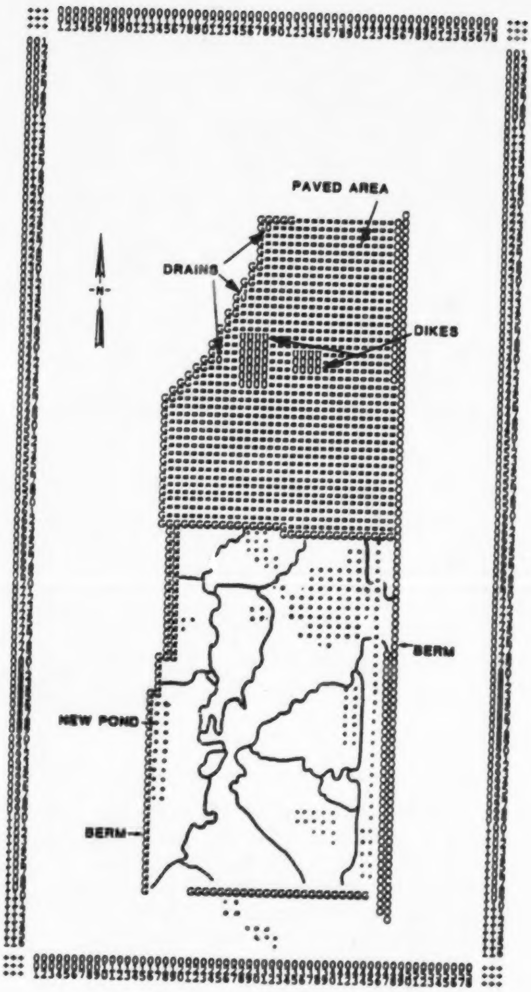
.... PK/BL TANKS-UNK 10 CELLS
//// VERT TANKS-UNK 55 CELLS
CASCADE PONDS 36 CELLS
:::: TANKS-UNK 6 CELLS
XXXX TANKS-UNK 2 CELLS
0000 PLATING TANKS 11 CELLS
1111 FLUE DUST 79 CELLS
2222 HORIZ TANKS-UNK 28 CELLS
3333 BARRELS W/PLAST 37 CELLS
4444 WASTE PILE-UNK 23 CELLS
5555 BARRELS-UNK 5 CELLS
6666 LEAD & WOODCHIPS 42 CELLS
7777 BARRELS-UNK 63 CELLS
8888 DEBRIS 23 CELLS

FIGURE 4 Waste features within the drainage basins of the Western Processing sites are identified (July 1983).


PAVED AREA
DRAINS
DIKES
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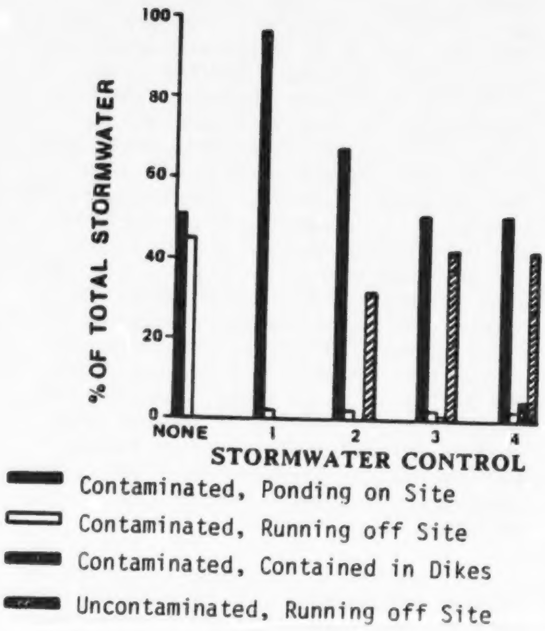
BERM
NEW POND
BERM

FIGURE 5 The effects of Remedial Action Alternative 4 on drainage patterns are examined for the Western Processing Site.



1 berms around site
2 pave northeast corner
3 pave northern half dikes around fuel tank
4 dikes around all storage tanks

FIGURE 6 Final stormwater disposition of all remedial alternatives are compared.

These results were obtained by simply altering the input topography map and applying the same cartographic modeling routines that were used in the first analysis. They would be useful in the evaluation or design of remedial action alternatives.

3. Excavation Volume Estimates and Site Regrading

The final demonstration analysis conducted in this study involved traditional engineering calculations. Excavation volume estimates and basal areas were developed for the major waste piles. Those materials were then "removed" by cartographic modeling techniques and the slope of each cell in the resulting surface was determined. The slopes were compared to those of the original site features and an assessment of erodibility in each case was made. The excavation scenario was continued below grade at one-foot intervals down to the water table. Excavation estimates are summarized in Table 4. These types of calculations are readily obtainable by the GIS and would be useful in assessing, negotiating, designing, and estimating costs for remedial actions.

TABLE 4. EXCAVATION ESTIMATES (4)

Excavated Waste Material	Volume (yd^3)
Flue Dust Pile	3525
Lead and Wood Chips Pile	3167
Gypsum Pile	3683
Contaminated Soils to 19 ft. AMSL*	117810

* approximate water table (3)

PROBLEMS ENCOUNTERED

Few problems were encountered in the demonstration analyses using the "MAP" program, once an adequate data base was established. However, several factors combined to make the creation of the spatial data base more time consuming than was initially anticipated. They were as follows:

* Difficulties in obtaining access to the data
* The applicability of available data to higher levels of analysis
* The absence of a master site map
* The age and response capabilities of available EPA equipment

In order to efficiently utilize a GIS there is a clear need for the EPA to develop advanced information processing and data management capabilities. Project continuity, access to existing data and the integration of study results from different media, contractors, disciplines, and time periods must be facilitated. EPA's current data management practices are such that the incorporation of a GIS into an ongoing project requires substantial input of resources. However, applying the GIS format at the earlier stages of a project would greatly facilitate data management efforts and could be accomplished with a modest expenditure of resources. Specific recommendations are:

* Require all site studies to conform to a master site map and all data to be collected in a location-specific format
* Adopt standards of quality for survey design and data collection
* Upgrade hardware and interactive computing capabilities

These requirements offer advantages in quality control, quality assurance, and efficiency regardless of the data analyses or management system employed.

RESULTS

The example analyses completed in this study successfully demonstrated many of the capabilities and usefulness of a GIS within the limitations of this data base. These particular analyses illustrate the GIS' ability to:

* Accept information from several type of data bases
* Initially characterize the site with traditional map reading techniques
* Use cartographic modeling to evaluate stormwater control remedial alternatives
* Perform traditional engineering calculations in estimating excavation volumes and site slopes

The function of a GIS in hazardous waste site application can be summarized in four general areas:

* Inventory and storage of site data in a common data format
* Data analyses either i) using the GIS or ii) preparing data for outside processing and returning results to the GIS
* Development and integration of external data bases
* Preparation of reports or special studies

This project did not demonstrate some aspects of the GIS important to sophisticated spatial data management and analyses -- the use of the GIS as an input-output device for other analytical systems. Many environmental models and statistical analysis packages require input data to be submitted in grid-based format. Sophisticated hydrologic models, the EPA UNAMAP series of air quality models, spatial interpolation routines such as kriging, and several statistical techniques operate with cartesian coordinate systems. The GIS can be used to develop input data for these routines and to accept, manipulate and further analyze results obtained from external programs. For example, the results of groundwater models could be analyzed in the context of local land use characteristics. A GIS can also be used to develop input information for external hardware devices. Data can be prepared for plotters and printers and as input to graphics display packages. The GIS' ability to store, combine, and manipulate the results of various site studies for output to external models and for display purposes could add a new dimension to hazardous waste site analyses.

In addition, continuing advances in digital cartography will greatly facilitate the application of GIS' in the next few years. Institutional data bases are being developed for direct input to a GIS. Many government agencies have developed GIS capabilities and several are digitizing their data files on a routine basis. Digital information is available for topography, roads and highways, utilities, soils, surface hydrology, vegetation cover, geology, and landuse. With these types of data a GIS can make rapid assessments of site suitability with regard to environmental conditions, surrounding land uses, and number of people affected by hazardous waste facilities. GIS' can be used to quickly assess site characteristics for disposal locations under RCRA authority. The development of standardized data bases by the USGS and the US Bureau of Census and the explosion in information and data processing capabilities will make it likely that GIS applications will become commonplace in the next decade.

ACKNOWLEDGEMENTS

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Sincere appreciation goes to the staff members of the Land Pollution Control Division at HWERL, Cincinnati, Ohio, EPA Region X, Seattle, Washington, the Environmental Monitoring Systems Laboratory (EMSL), Las Vegas, Nevada, and the UC Center Hill Facility, Cincinnati, Ohio who provided guidance, data and facilities.

Special thanks go to the following individuals: Douglas C. Ammon, HWERL Technical Project Monitor, for his support and advice in the conception and implementation of this project; John Barich, EPA Region X, for his special interest in the project and insights about the Western Processing site; and Martha Lambert, University of Cincinnati, for her support and assistance at the Center Hill Facility.

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QUALITY CONTROL OF HYDRAULIC CONDUCTIVITY AND BENTONITE CONTENT DURING SOIL/BENTONITE CUTOFF WALL CONSTRUCTION

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ABSTRACT

Soil/bentonite backfilled cutoff walls (slurry walls) have been used over the past several years to contain and isolate hazardous wastes. While may cutoff walls have been installed, their effectiveness and quality of construction have not been studied or documented to any extend. This paper reports on the first phase of testing at the Gilson Road site, Nashua, New Hampshire. The objective of this phase was to evaluate quality control measures for the determination of bentonite content and hydraulic conductivity. This paper specifically addresses methylene blue titration for measurement of bentonite content and the API fixed ring permeameter for evaluation of hydraulic conductivity. The accuracy, applicability and practicability of these tests as field quality control measures are discussed.

INTRODUCTION

Soil/bentonite backfilled cutoff walls (slurry walls) have successfully been used as an adjunct to pumping for excavation dewatering and general groundwater control for over forty years. Specifications governing this work typically fell under the dewatering section of construction documents and, as such, were performance oriented. In many cases, success was judged solely on the ability of the barrier to limit flows to quantities that could be reasonably pumped. More recently, soil/bentonite (S/B) cutoff technology has been gaining acceptance as a tool for remediation of US EPA National Priority List (NPL) sites, as well as private industrial sites. For these more critical applications, leakage rages which were considered insignificant now represent unacceptable performance with respect to hazardous waste containment.

The successful construction of a hazardous waste cutoff wall for containment is still largely dependent on the quality of the contractor's workmanship and as such, specifications remain performance oriented. However, in recognition of the difficulties involved in performance verification and the consequences of detecting substandard construction only after wall completion, contract documents are beginning to include minimum design standards. These center around minimum bentonite and fines content (see Ayres et al. 1983). This allows quality control (QC) testing to be executed in the field during construction as a check on attainment of the minimum standards specified (Figure 1).

Quality control testing for the Gilson Road (NPL site) cutoff wall was consciously planned from design through specification preparation and then executed in the field to effectively guide the contractor towards a successful S/B cutoff wall installation. New S/B backfill QC techniques, including methylene blue titration and API fixed ring permeation, were utilized for bentonite content and hydraulic conductivity determinations, respectively. This work demonstrated the applicability and practicability of these new methods for field control of S/B backfills and thus led the US EPA to fund research into the refinement of the procedures and evaluation of their precision and accuracy. This work represents Phase I of a larger

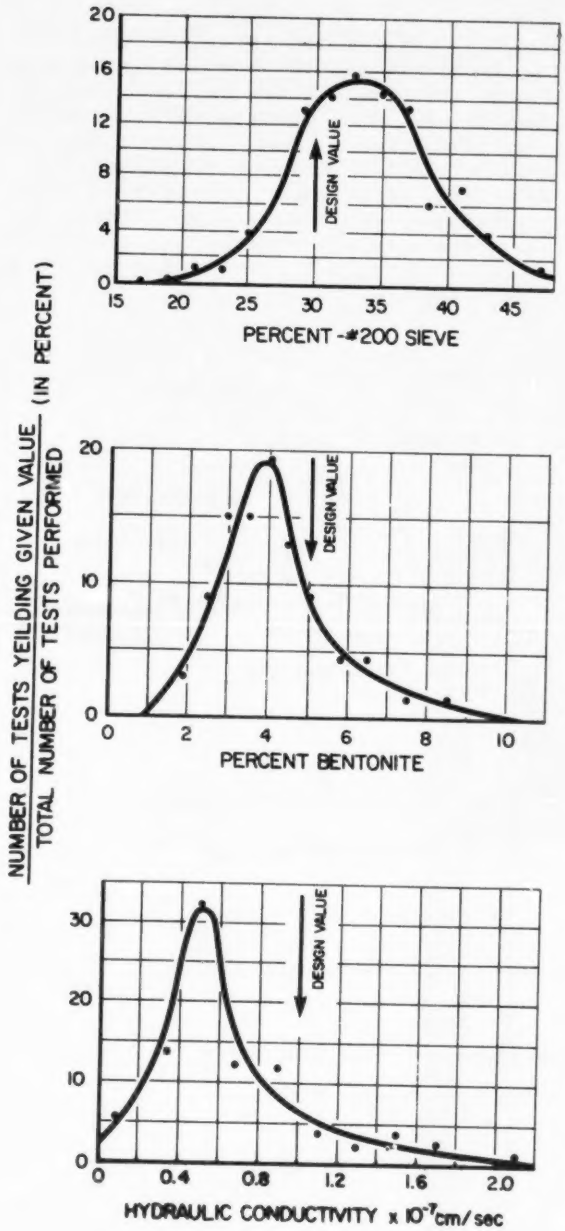


Figure 1. Quality Control Data Obtained During Construction of the Gilson Road Cutoff

effort funded by a cooperative agreement between the New Hampshire Water Supply & Pollution Control Commission (NHWSPCC) and US EPA which also includes post-construction barrier continuity verification via piezocone techniques, undisturbed sampling/lab testing, and pump testing. Subsequent phases of the research are currently ongoing with the overall objective of producing the basis for general QC and verification testing guidelines for S/B cutoff walls.

EVALUATION OF BACKFILL HYDRAULIC CONDUCTIVITY

Fixed ring equipment was used to provide hydraulic conductivity QC data during construction of the Gilson Road cutoff wall. This inexpensive, easily operated equipment allowed field testing of numerous samples prior to permanent incorporation of the backfill into the cutoff. The one day availability of hydraulic conductivity data allowed real time quality control of the construction process. In light of the high degree of usefulness and practicability of the fixed ring hydraulic conductivity testing as shown on the Gilson Road project, a R&D testing program was undertaken to:

1. Assess the actual accuracy and precision of the test
2. Based on No. 1 above, evaluate the validity of the QC data collected on the Gilson Road project, and
3. Refine and standardize the test procedure for incorporation in general guidelines for QC testing of S/B cutoff walls.

API Fixed Ring Test

Various equipment configurations based on fixed ring confinement of a test sample for evaluation of hydraulic conductivity have been in use for many years. This equipment is discussed in the U.S. Army Corps manual (1970) and Lambe (1951). In most cases, however, the equipment is meant for laboratory use and not designed to withstand the "environment" associated with field testing. In an effort to standardize procedures around equipment which would survive repeated field use and also be readily obtainable, the conventional

American Petroleum Institute (API) filter press was adapted for use as a fixed ring permeameter (see Figure 2). As a filter press, this equipment is already routinely used on cutoff wall projects to test the filter cake forming capacity of bentonite slurries used for trench stabilization.

In basic terms, API hydraulic conductivity testing involves placing a sample in the cell, covering it with water, and forcing the water through the sample using pneumatic pressure. Once consolidation of the sample is complete, the steady state hydraulic conductivity is computed based on standard Darcy's Law relationships assuming 100 percent saturation. For typical S/B backfill samples, approximate values of hydraulic conductivity can be obtained in several hours and final values in less than 24 hours. As such, the procedure is ideal for real time quality control of the construction process. A detailed description of the standardized test procedure, which directly reflects the results of research efforts performed for this project, will be presented in a future report.

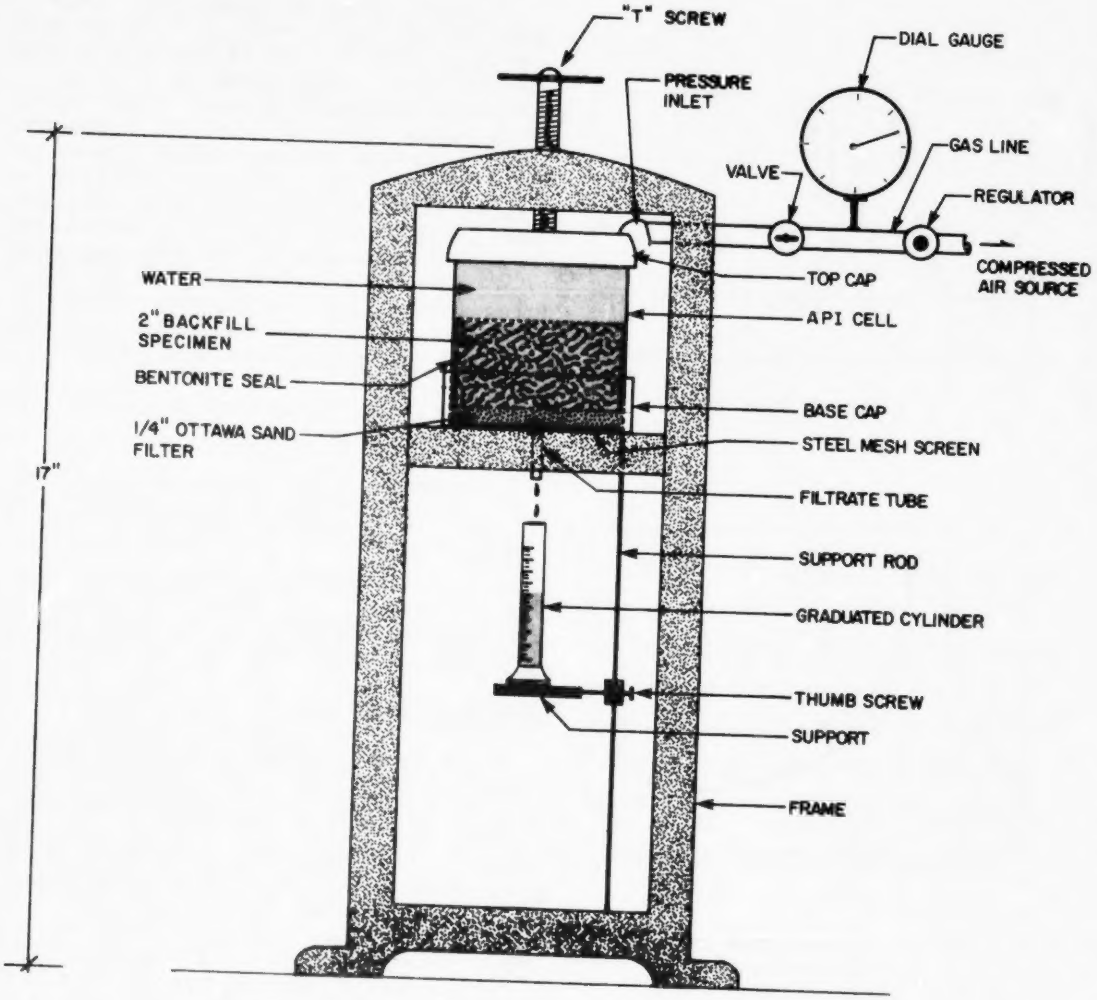

"T" SCREW
DIAL GAUGE
PRESSURE INLET
GAS LINE
VALVE
REGULATOR
WATER
TOP CAP
COMPRESSED AIR SOURCE
API CELL
2" BACKFILL SPECIMEN
BENTONITE SEAL
BASE CAP
1/4" OTTAWA SAND FILTER
STEEL MESH SCREEN
17"
FILTRATE TUBE
SUPPORT ROD
GRADUATED CYLINDER
THUMB SCREW
SUPPORT
FRAME

Figure 2. API Hydraulic Conductivity Apparatus

API Test Precision

The first level of investigation was directed towards determining if equipment and procedures as simple as the API based QC testing performed on the Gilson Road project would provide repeatable hydraulic conductivity values. The degree to which the precision of a test procedure can be established, however, is highly dependent on how identical the test specimens are prepared. This is problematic for API hydraulic conductivity testing in that a single specimen cannot be set up more than once. Therefore, multiple specimens are required to evaluate the degree of variability in the entire procedure, from set-up through permeation. Unfortunately, the coarseness of granular based backfills such as used on the Gilson Road project can result in a somewhat heterogeneous sample from which the individual aliquots are taken for the comparison testing. Therefore, uniform mixing, to the extent practical in the laboratory, was emphasized during sample preparation. However, it is likely that some degree of heterogeneity still existed among the comparison samples.

Five batches of the Gilson Road backfill were selected from record samples obtained during construction. Selection emphasized encompassing the full range of gradations and hydraulic conductivities encountered during construction of the actual cutoff wall. After thorough remixing in the laboratory, three "identical" aliquots of each sample were tested at a gradient of 40 (driving pressure of 3 psi). The results are presented in Table 1. The data indicate a worst case precision of ±57 percent within one standard deviation of the mean value for any group of three tests. Although this initially appears as a large degree of scatter, it is actually quite small when compared to the five orders of magnitude (100,000 times) reduction in permeability of the initial in-situ soils. Further inspection of the data also reveals that in all five groups of tests,

TABLE 1

API HYDRAULIC CONDUCTIVITY TEST; PRECISION SERIES

SAMPLE	HYDRAULIC CONDUCTIVITY X 10-7 in cm/sec Individual Splits of Each Sample	Mean	One Standard Deviation	STD. DEV. / MEAN in %
Sta. 0+05	0.48 0.38 0.46	0.44	±0.05	11%
Sta. 7+75	4.02 4.02 4.44	4.16	±0.24	6%
Sta. 14+75	0.80 1.86 0.79	1.15	±0.61	53%
Sta. 20+25	0.63 0.25 0.25	0.38	±0.22	57%
Sta. 35+00	0.69 0.28 0.38	0.45	±0.21	47%

at least two of the three tests in each group agreed to ±10 percent within one standard deviation of the mean. Hence, some of the scatter reflected in the worst case precision of ±57 percent may be attributable to the effect of less than identical samples as previously explained. This hypothesis is supported by field data obtained during subsequent QC testing for other cutoff walls utilizing primarily clayey mixes where the probability of obtaining identical samples is significantly enhanced.

API Test Accuracy

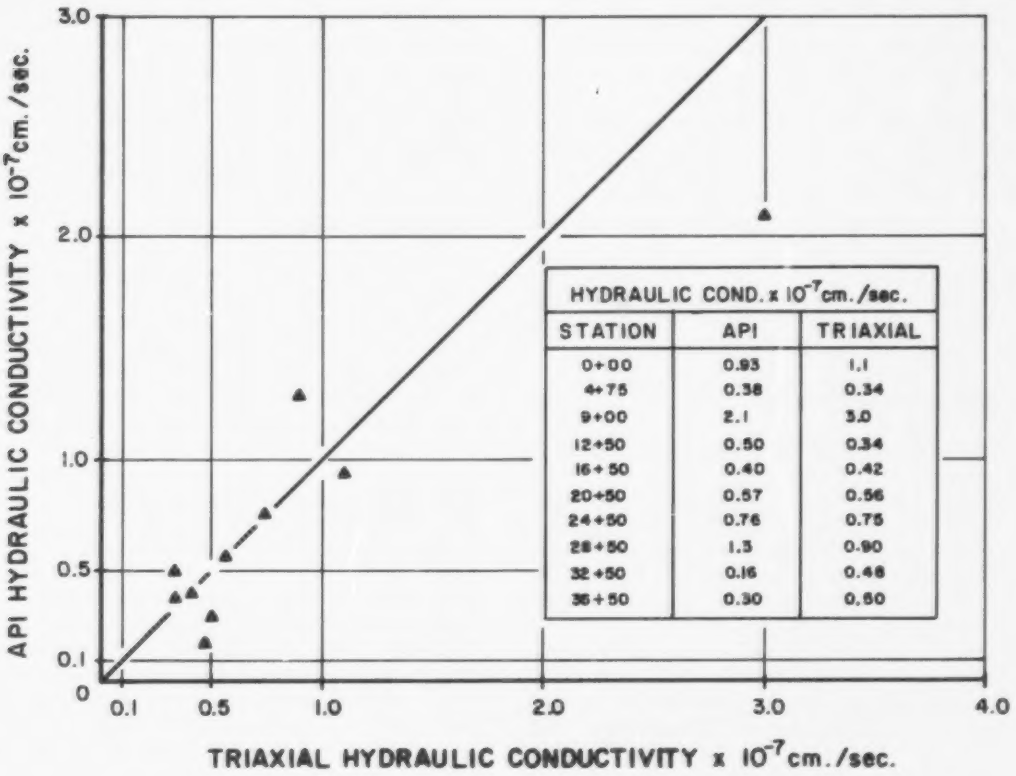
To evaluate the accuracy of a specific test procedure, it is imperative to know the "true" value of the parameter of interest for the specimen being tested. This is not possible for hydraulic conductivity due to its dependence on the method of measurement used. The value of hydraulic conductivity adopted as the "absolute" value to which all other values are compared in this work was derived via triaxial testing. Triaxial testing, however, can also yield a wide range of values for the same S/B backfill sample. This variability stems primarily from the lack of standardization of physical testing parameters such as state of stress, degree of saturation, gradient, etc., to which the sample is highly sensitive (see Schulze et al., 1984). The accuracy evaluations made herein are therefore based on "absolute" triaxial values in which the physical testing parameters were standardized to model in-situ conditions at the top of the cutoff wall (upper 5 to 10 feet). As such, these "absolute" values of hydraulic conductivity would be conservative (high) relative to those expected for the same sample tested under conditions simulating greater depths or incomplete saturation. The philosophy of modeling the worst case condition of "top of wall" recognizes the indeterminacy of the final vertical location of a specific batch of backfill represented by a QC sample taken prior to backfill placement.

During construction of the Gilson Road cutoff wall, ten samples of the backfill were obtained for triaxial testing. These samples were split and one portion was also field tested in the API apparatus as part of the QC program. Therefore, to the degree that the samples taken were homogeneous, the value of hydraulic conductivity was determined by both triaxial and API procedures on "identical" specimens. These data are presented in Figure 3. The data indicate that the API values match the triaxial values to within ±66 percent as a worst case. This error band not only encompasses the accuracy of the API test but also incorporates errors associated with heterogeneity between "identical" samples and precisional errors inherent in both the API and triaxial procedures. As such, the data indicate a very good level of agreement between the simple field API test and the more sophisticated laboratory triaxial test.

Field Use of the API Test

Based on the research presented herein, and the field data from actual cutoff wall projects undertaken to date, the API test appears to yield reliable and accurate values of hydraulic conductivity. However, it is considered prudent to run a limited number of triaxial tests for each project to verify the site specific accuracy of the API procedure. The initial check between API and triaxial data should be made during the design phase of the project when backfill component selection and proportioning is established based on laboratory testing. This design testing should then be followed by triaxial testing of actual backfill samples obtained during construction.

Although a high degree of additional backfill mixing is required for the API/triaxial comparison samples, mixing of field samples taken for QC testing in excess of that achieved during construction is highly undesirable. This directly reflects the dependence of hydraulic conductivity on the degree of backfill component blending to a scale commensurate with that of interstitial pore size. Data from triaxial testing of post-construction, undisturbed samples indicate that up to an order of magnitude reduction in hydraulic conductivity can occur if the originally tested undisturbed samples are mixed thoroughly and retested. Therefore, samples for QC should be taken and placed in the test chamber with as little additional mixing as possible while still removing air voids. Failure to observe this procedural "detail" can



HYDRAULIC COND. x 10^{-7}cm./sec.		
STATION	API	TRIAXIAL
0+00	0.93	1.1
4+75	0.38	0.34
9+00	2.1	3.0
12+50	0.50	0.34
16+50	0.40	0.42
20+50	0.57	0.56
24+50	0.76	0.75
28+50	1.3	0.90
32+50	0.16	0.48
36+50	0.30	0.60

Figure 3. Comparison of API vs. Triaxial Hydraulic Conductivity

result in artificially low values of hydraulic conductivity. It also follows directly from this discussion that both the API and triaxial data sets obtained for test comparison, do not qualify as QC data. The sole purpose of these tests is to demonstrate the site specific validity of the API procedure.

The gradient (driving pressure) used during API QC testing should be that value which yields a hydraulic conductivity closest to that derived from triaxial testing. (The gradient used for the triaxial testing is commensurate with "top of wall" stresses as previously discussed). It is emphasized that significantly higher gradients should not be used in an effort to increase the flow per unit time so as to make effluent volume measurements easier. High gradient results in consolidation of the sample to levels no longer commensurate with "top of wall" stresses. As such, artificially low values of hydraulic conductivity can be obtained. Furthermore, high gradient can lead to piping and thus unrealistically high values of hydraulic conductivity not representative of the future performance of the wall.

EVALUATION OF BACKFILL BENTONITE CONTENT

In the past, field measurements of bulk quantities of backfill soils and bentonite have been used to determine bentonite percent by applying basic weight/volume relationships. This method is approximate due to the limited accuracy of the bulk weight/volume estimates, and optimistically assumes the bentonite is evenly mixed throughout the backfill. Thus, bulk measurements can be used as a gross check on the percentage of bentonite in the backfill, but are inadequate for quantitative field control. As an alternative, the use of conventional hydrometer test procedures have been attempted for bentonite determination. Inherent limitations in the test with respect to accuracy, and completion time, however, restrict its use for backfill QC testing. A laboratory testing program was therefore carried out to investigate the applicability of the quicker and potentially more accurate methylene blue test for determination of percent bentonite in S/B backfill.

Methylene Blue Titration

In a suitable aqueous environment, such as an electrolytic clay-water suspension, clay materials are associated with naturally occurring absorbed cations of a fixed total charge (typically sodium, calcium, magnesium, or potassium). These cations are either retained by the clay or may be exchanged for other cations subsequently introduced into solution. The degree to which this exchange may occur, referred to as cation exchange capacity, increases with the amount and activity of the clay in suspension. The determination of bentonite content therefore relies on the proportionality between the amount of methylene blue cations exchanged and the amount of bentonite in the backfill sample tested. The quantity of methylene blue cations exchanged is determined by titration.

The titration procedure involves the incremental addition of methylene blue solution to a suspension consisting of the backfill sample and a predetermined volume of distilled water and dispersant until the cation exchange capacity of the suspension is exceeded (end point). Methylene blue consumption at the end point is then used to determine the dry weight of bentonite in the sample via a pre-established correlation. Although the titration volume actually corresponds to the dry weight of bentonite, the data will be presented herein as an equivalent bentonite percent in the sample. This format is used in recognition of the ultimate objective of standardizing this procedure for QC of S/B backfill mixes.

The methylene blue test is well suited for quality control of S/B backfills due to the relatively short testing time (approximately 30 minutes per test), low cost, and minimal laboratory requirements. A detailed description of the standardized test procedure, which directly reflects the results of research efforts performed for this project, will be presented in a future report.

Testing Program

Evaluation of the accuracy and precision of methylene blue titration when used as a QC test for S/B backfill was separated into four distinct phases. Phasing was implemented to progressively assess the impact attributable to increasingly more complicated matrices accompanying the bentonite. These matrices encompassed the simplest case of distilled water only to more realistic combinations approximating actual backfill samples including natural soils with non-bentonite related cation exchange capacity and interstitial water contaminated with hazardous wastes. The applicability of linear proportionality, matrixing effects and superposition were thus investigated. Testing accuracy was assessed through comparisons of measured bentonite content with that actually added to the laboratory prepared samples. The four phases were:

- Bentonite in a distilled water matrix;
- Bentonite in an Ottawa sand and distilled water matrix;
- Bentonite in a contaminated water matrix;
- Bentonite in an off-site borrow, trench spoil and distilled water matrix.

Distilled Water Matrix (Phase I)

Bentonite in a matrix of distilled water and dispersant was chosen for the first phase of testing to eliminate complicating reactions which might otherwise be induced by additional backfill components. This, the simplest possible matrix, was used to assess the proportionality and inherent accuracy/precision of the methylene blue titration for bentonite content. Three to four aliquots each of ten slurries containing increasing bentonite amounts from 0.0 to 0.9 grams were prepared. The bentonite amounts selected correspond to 10 gram backfill samples containing 0 to 9 percent bentonite by dry weight. Standardization of a 10 gram sample size reflects sampling, testing, and typical backfill gradation considerations. The same bentonite was used throughout the testing and conformed to the American Petroleum Institute's API grade, minimum 91 barrel yield.

For each of the 36 individual tests, the volume of methylene blue solution added to reach the endpoint of titration (completion of cation exchange) was plotted versus the known percent bentonite referenced to a 10-gram sample (see Figure 4). The trend portrayed by the data not only demonstrates proportionality but also indicates a linear relationship between methylene blue titration volume and percent bentonite. Linear regression analysis performed on the data yields a line referred to as the "bentonite calibration curve" which fits the data set to a high degree as reflected by a coefficient of determination equal to 0.999. Hence, given a slurry sample containing an unknown amount of bentonite, it would be expected that the titration volume in conjunction with the previously established calibration curve would yield the correct percent bentonite to within ±0.2 percent with a 95 percent degree of confidence.

The bentonite/distilled water samples used to establish the basic relationship between titration volume and percent bentonite excluded all other possible backfill components. This "best case" condition is therefore expected to yield

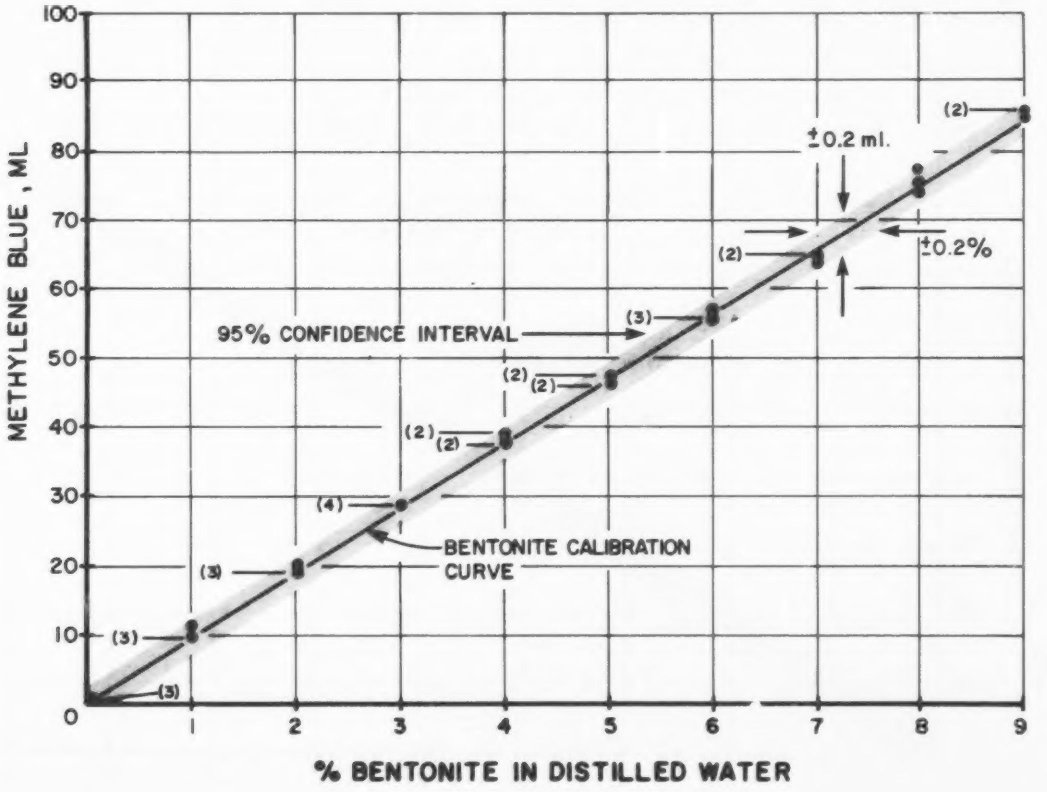


Figure 4. Titration for Bentonite Content in Distilled Water Matrix

the highest degree of accuracy. The curve defined by this data is the baseline calibration against which the results of subsequent tests on more realistic backfill samples are compared and testing accuracy assessed.

Ottawa Sand/Distilled Water Matrix Phase 2)

For the second phase of testing, known amounts of bentonite were added to a matrix composed of Ottawa sand, distilled water, and dispersant. These samples were one degree more complex than those used for Phase I testing and also one step closer to a typical backfill sample. Ottawa sand was selected in that it is almost entirely quartz with no clay minerals present. As would be expected based on cation exchange principles, quartz, and thus the Ottawa sand, has no exchange capacity. The specific objective of this testing phase was therefore to determine whether the presence of backfill components having no cation exchange capacity would interfere with the bentonite/methylene blue cation exchange. This interference, referred to as "matrixing effect," would include any physical, chemical, or electrical phenomena which alter the surface activity of the bentonite clay mineral.

Three identical samples were prepared individually at each of five bentonite contents; 0, 2, 4, 6, and 8 percent. Each sample contained appropriate dry weights of Ottawa sand and bentonite to yield 10 gram "backfill" samples with the specified bentonite percentage. These samples were added to the standardized volume of distilled water/dispersant and slurried for testing. For each of the 15 tests, the methylene blue titration volume at end point was plotted versus the known bentonite percentage (Figure 5). The previously determined calibration

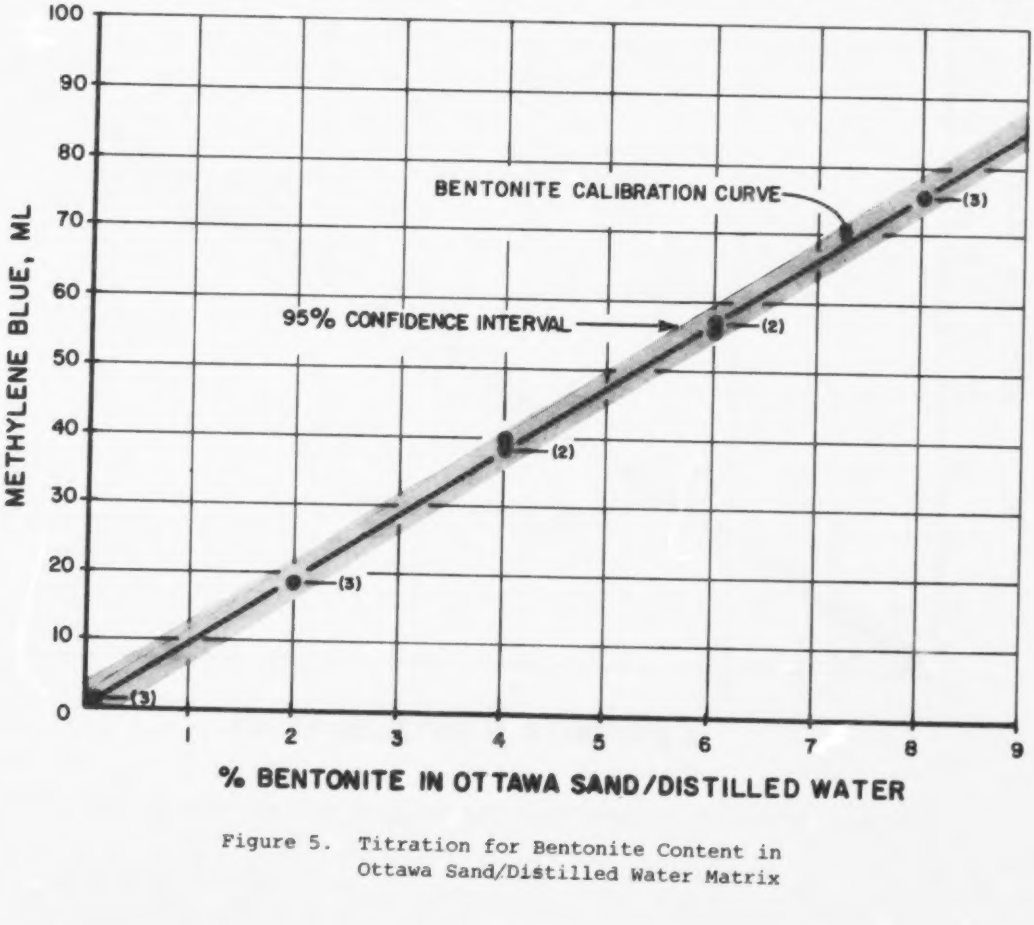


Figure 5. Titration for Bentonite Content in Ottawa Sand/Distilled Water Matrix

curve and 95 percent confidence interval (from Figure 4) are also presented with the data for comparison. The data indicate that, within the ±0.2 percent error band established via the 95 percent confidence interval, the Ottawa sand has no effect on the methylene blue/bentonite cation exchange process. Therefore, matrixing problems are not probable due to the presence of quartz based backfill components with no cation exchange capacity.

Contaminated Water Matrix (Phase 3)

During methylene blue testing, backfill samples are, in all cases, mixed with distilled water to create a suspension which is then titrated. Hence, the water used during the testing is always contaminant free. However, contaminants may be added to the suspension via the backfill sample itself, particularly where the cutoff wall is excavated through contaminated portions of the aquifer. To assess the possible matrixing effects of these contaminants on the accuracy of the methylene blue test, a worst case scenario was utilized for sample preparation. In addition to incorporation of contaminants via the backfill, contaminated water was used to form the test suspension instead of distilled water.

Contaminated water from three different sources on the Gilson Road site were obtained for this phase of the work. The first two, designated well water and stream water, exhibited significant hardness and were sources actually used during construction for slurry makeup water. The third, and most contaminated source, was leachate containing volatile organic solvents taken from monitoring installations on the site. Backfill samples were prepared at 0, 2, 5, and 8 percent bentonite in a manner similar to Phase I testing. Two aliquots were tested for each bentonite content using the stream and well water (total of 16 samples). One series was performed using leachate at the four bentonite contents (4 samples). The data, presented in Figure 6, indicate that the contaminants tested had no significant effect on the test results. All but one of the individual data points fall within ±0.2 percent of the original bentonite calibration curve. As such, the initial calibration accuracy of ±0.2 percent with a 95 percent confidence level was maintained in spite of the Gilson Road contaminants introduced into the matrix tested.

For volatile organic contaminants, noninterference would be expected inasmuch as they are non-ionic and therefore should not exhibit a high cation exchange capacity. In addition, the backfill sample is dryed in an oven at 105°C and the backfill/water/dispersant matrix is boiled prior to titration. The temperature involved in the above procedural steps would be expected to remove the volatiles. For other sites, particularly those containing highly ionic contaminants, the cation exchange potential of the species present should be compared to that of methylene blue with respect to affinity for the bentonite clay mineral. In cases where high non-volatile solvent concentrations exist, reduced solvent (water) polarity effects, as they relate to methylene blue solubility, should also be investigated. This can be done on a theoretical basis or by utilizing the simple testing procedure described herein. It should be recognized however that this testing procedure can yield "false positives" in that the quantity of contaminants introduced into the matrix far exceeds that which is likely to be added via an actual backfill sample.

Gilson Road Backfill Matrix (Phase 4)

The fourth phase of testing employed prepared samples duplicating the backfill used for the Gilson Road project. The Gilson Road backfill consisted of approximately equal proportions of trench spoil (sand and gravel) and off-site borrow (sandy silt) with 3 to 5 percent bentonite added. The objectives of this series of tests were to 1) evaluate the impact of a cation exchange capacity attributable to non-bentonite backfill components on the determination of percent bentonite in the backfill; 2) determine if the bentonite calibration could be corrected to account for non-bentonite cation exchange capacity utilizing superposition; and 3) based on the above, evaluate the accuracy of the methylene blue test results previously obtained for the Gilson Road project.

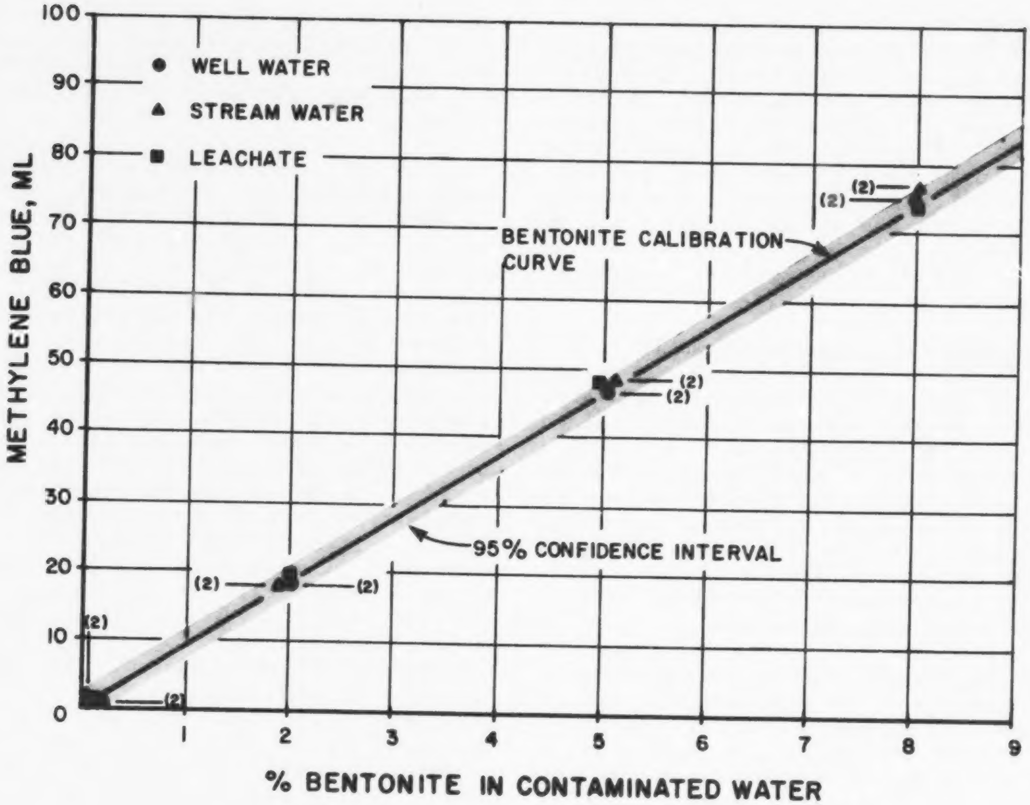


Figure 6. Titration for Bentonite Content in Contaminated Water Matrix

The on-site soils (trench spoil component of the backfill) used for testing were obtained during a post-construction boring program. The ten samples were mixed with distilled water and dispersant in a manner similar to Phase II testing except that no bentonite was added. The on-site soils exhibited average cation exchange capacities equivalent to 0.2 percent bentonite with a range of 0.1 to 0.4 percent. Nine samples of the off-site borrow were also selected from record samples taken during construction. Eight of these samples yielded bentonite equivalents of 0.6 to 0.7 percent with one sample yielding 0.4 percent. This preliminary testing for the exchange capacity of individual non-bentonite backfill components provided a basis for prediction of the bias expected relative to the bentonite calibration previously developed. An anticipated offset equivalent to 0.4 percent bentonite was derived using a weighted average of the individual exchange capacities based on the relative amounts of borrow and trench spoil in the backfill.

Fifteen splits of backfill were prepared containing equal proportions of the on-site soil and the off-site borrow. Bentonite was added to the resulting mixture (a silty sand and gravel) to obtain three samples at each of 5 bentonite contents; 1, 3, 5, 7, and 9 percent by dry weight. The samples were titrated with methylene blue solution until endpoint and the data plotted on Figure 7. Once again the data exhibited a linear trend, but, as expected, also demonstrated bias reflecting greater exchange capacity than attributable to the previously predicted 0.4 percent non-bentonite derived off-set on the original calibration curve yielded a "corrected calibrated curve" presented as a dashed line on Figure 7. The original bentonite calibration curve developed during Phase I testing is also presented for comparison.

Although the corrected calibration does not exactly match the best fit line through the actual data, it does agree to ±0.3 percent bentonite. These data therefore support the validity of superposition in that the small discrepancy obtained corresponds to inaccuracies incurred by averaging the data upon which the corrected calibration curve was based. This error band represents less than ±6 percent of the 5 percent specified bentonite and verifies that the data amassed during the cutoff wall construction at Gilson Road was in fact sufficiently accurate for field QC use.

Field Use of the Methylene Blue Test

The laboratory program described herein was used to assess the accuracy of the methylene blue test and was therefore structured to measure the unknown quantity of methylene blue required to titrate backfill samples with known bentonite contents. The data thus developed demonstrated the utility and accuracy of the test for field QC in cases similar to the Gilson Road project. During actual use for QC in the field however, the methylene blue consumption would be measured and used to determine the unknown bentonite content in the backfill via pre-established calibrations. Thus, the testing methodology used in field applications initially would follow a logic similar to the laboratory program for establishment of a calibration curve, but thereafter utilizes methylene blue consumption as the independent variable

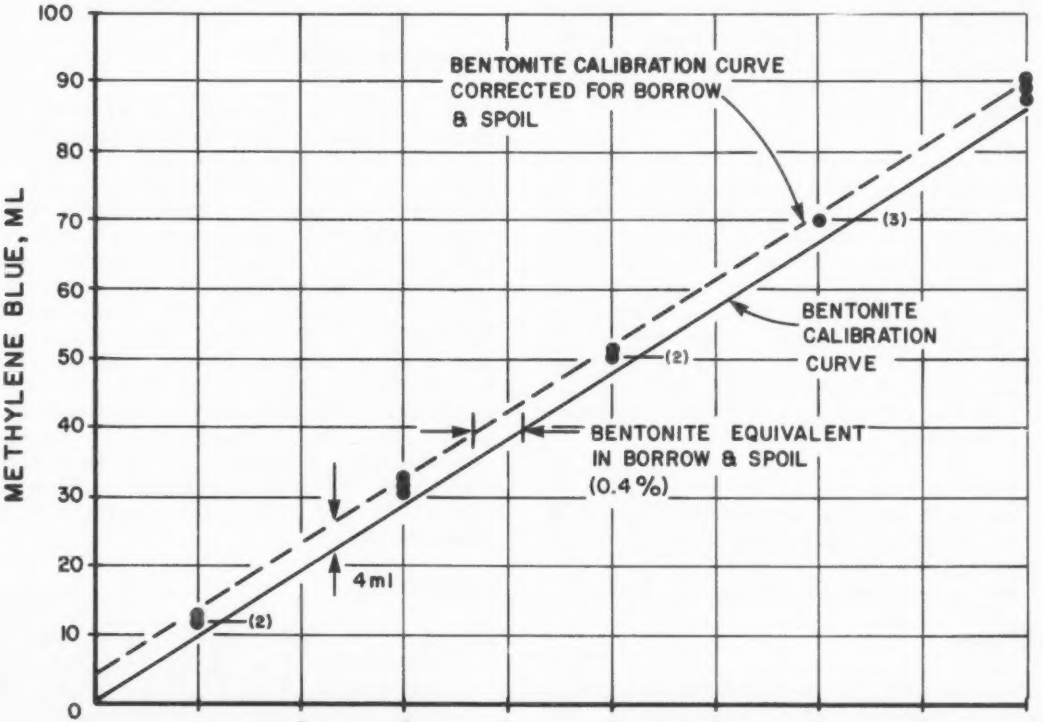


Figure 7. Titration for Bentonite Content in Gilson Road Backfill Matrix

and bentonite content as the dependent variable. Implementation of the methylene blue test for quality control therefore consists of the following procedure:

1. Establish a bentonite calibration curve representing the relationship between methylene blue consumption and bentonite content for bentonite/distilled water suspensions. The procedure would be similar to the Phase I testing described herein except that the methylene blue consumption would be plotted on the x-axis as the independent variable.

2. Determine if contaminants are likely to be present in the backfill samples and evaluate the probable effect on the calibration.

3. Determine average cation exchange capacity (methylene blue consumption) of the non-bentonite backfill soils combined in the same proportions they will be mixed in the field. Use the bentonite calibration curve to determine the bentonite equivalent for this volume.

4. Offset the initially determined bentonite calibration curve to account for the bentonite equivalent of the non-bentonite soils and/or contaminants to establish a field calibration curve. Note that multiple field calibration curves may be required if significant variation in the offset values are anticipated due to variations inherent in the site geology or construction procedures.

5. Perform the methylene blue test on backfill samples and enter the field calibration curve with the volume of methylene blue consumed at the end point to determine bentonite content.

CONCLUSIONS

The work presented herein was directed towards verification of the precision, accuracy, and practicality of two new tests for QC of S/B backfills. This is part of a larger endeavor focused on the development of standardized QC guidelines concordant with a strategy of real time field testing to ensure attainment of an effective barrier during its construction rather than resorting to post-construction performance verification based litigation. As based on this work, it can be concluded that the methylene blue test for percent bentonite and the API hydraulic conductivity test both satisfy the four basic criteria required of quality control testing:

- They are both inexpensive and thus allow a statistically significant number of tests to be performed.
- They are simple thus lending the procedures to execution in the field rather than a remote laboratory.
- They are quick and therefore can provide the timely data necessary for real time quality control and,
- They are sufficiently precise and accurate to yield a meaningful representation of the expected post-construction performance.

ACKNOWLEDGEMENTS

The research and development work presented herein was associated with the Gilson Road NPL site and as such was sponsored under a cooperative agreement between the State of New Hampshire Water Supply and Pollution Control Commission and the U.S. EPA. The authors would therefore like to express appreciation to Michael Donahue, Thomas Roy, and Paul Heirtzler of the State of New Hampshire and Stephen James, Walter Grube, and John Herrmann of the U.S. EPA for their continuing cooperation and assistance. In addition, the authors would like to acknowledge Katrina Bikis and Donald Schulze without whose research contributions this paper would not have been possible.

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CONTROL OF FUGITIVE DUST EMISSIONS AT HAZARDOUS WASTE CLEANUP SITES

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ABSTRACT

Three field studies were performed to determine the effectiveness of dust control strategies at hazardous waste sites. A total of 12 dust suppressants were tested to determine the effectiveness of fugitive dust control against wind erosion from exposed areas. Based on a tracer sampling protocol, the suppressants were 100 percent effective for 1 to 4 weeks after application, with declining control efficiencies thereafter. The second wind erosion field study was an evaluation of the effectiveness of windscreens and windscreen/dust suppressant combinations in controlling fugitive dust from storage piles. Based on 82 tests, the windscreen showed no consistent significant level of control of particles ≤ 10 micrometers. The wind erosion emission rate of particles ≤ 10 micrometers is fairly constant at windspeeds above a threshold of about a 7 mph average. The windscreen may control larger particles more effectively, but these particles are less of a threat because they exceed the inhalable size and are not carried long distances in the wind. Chemical stabilizers were superior in performance to the windscreen. The third field study, investigating active cleanup emissions, consisted of testing fugitive dust control measures applicable to the use of a front-end loader to load dirt into a truck. Control efficiencies of 41 to 77 percent were measured for area spraying and spray curtains. In general, chemical dust suppressants are often wastes or by-products themselves. Before applying a dust suppressant, the user should analyze the dust suppressant for toxic substances and test for reactivity between the dust suppressant and the contaminated soil.

INTRODUCTION

Spills, waste disposal, and various industrial operations can result in the contamination of land surfaces with toxic chemicals. Soil particles from these areas can be entrained into the air, transported offsite via the wind, and result in human exposure by direct inhalation. Indirect exposure can result when particulates are deposited in agricultural fields, pastures, or waterways and enter the human food chain.

Contaminated soil can be reentrained in the air by three basic mechanisms:

- Wind erosion
- Reentrainment by moving vehicles (rubber-tired or tracked-vehicles) on soil or paved/unpaved roads
- Active cleanup (movement of soil by dozers, loading by front-end loaders, etc.)

These three mechanisms can act individually or in any combination.

PURPOSE

Very few data are available to assist in developing dust control programs at cleanup sites. Field testing of dust control effectiveness has been limited largely to vehicle reentrainment of dust from paved and unpaved roads. The intent of this project was:

- To perform field demonstrations of several products to determine their effectiveness for controlling dust.
- To prepare a handbook on state-of-the-art methods of dust control.

Three field studies were performed. The first involved the testing of 12 dust suppressants to determine how effectively they control fugitive dust resulting from wind erosion of exposed areas. The second wind-erosion field study was an evaluation of the effectiveness of windscreens and windscreen/dust suppressant combinations in controlling fugitive dust from storage piles. The third field study, investigating active cleanup emissions, consisted of testing fugitive dust control measures applicable to the use of a front-end loader to load dirt into a truck.

A Dust Control Handbook was prepared which included discussions of the following for each of the three basic reentrainment mechanisms:

- Identification of dust producing points
- Principles of control
- Product listing by name, address, telephone number, dilution, application rate, basic application method, and cost
- Detailed application procedures
- Product effectiveness

APPROACH

Control of Wind Erosion Dust Emissions From Exposed Areas

Tracer sampling was chosen as the methodology most closely paralleling the requirements of this study. A tracer was applied to the soil surface before a commercial dust suppressant product was applied to the plot. Any tracer-laden particles found later in the ambient air around the test plot indicated a failure in the integrity of the crust formed by the dust suppressant.

The test plots were located on a small farm near Cincinnati, Ohio. The soil type in the test plots was clay and quite uniform from plot to plot. The plots were spaced several hundred feet apart to eliminate cross contamination of the plots by the tracer. The plots were prepared by removing vegetation from 50 ft x 50 ft areas with a bulldozer and grading the plot to make it smooth. Application of zinc sulfate (water-soluble), the selected tracer, resulted in a surface concentration of about 300 ppm. After application of the tracer, the dust-control products were applied according to the manufacturer's recommendations. Table 1 describes each test plot. No control plots (tracer contaminated plots with no dust suppressant) were established. Emissions from the control plot would have likely contaminated other test plots at the site, resulting in a false indication of dust suppressant failure.

Particulates being removed from the plot by wind movement were sampled with saltation catchers and surface vacuum sampling. Four catchers (36-inch plastic tubes with a 2-inch-wide vertical slot that samples at 12- to 30-inch height intervals) were placed at each plot; they were oriented at the midpoint of each side, with the sampling slot facing the plot. Samples from the saltation catchers were taken about once each week during the test period and were analyzed for zinc by atomic absorption spectroscopy. An area of about 35 ft^2 was vacuumed each week from each test plot and placed onto a glass fiber filter. The material was removed from the filter and also analyzed for zinc.

TABLE 1. EXPOSED AREA TEST PLOTS

Name	Material cost/acre, dollars	Formulation	Application concentration*	Application rate
Soil Seal	$1450	Latex Acrylic Copolymer	3%	1.0 gal/yd^2
AMSCO-RES 4281	2350	Carboxylated styrene-Butadiene Copolymer	20%	0.6 gal/yd^2
Supac 8 NP	4840	Nonwoven Geotextile	8 oz/yd^2	12-ft rolls
Flambinder	62	Lignosulfonate	17%	0.5 gal/yd^2
Genaqua	484	Vinyl Acetate Resin	10%	0.2 gal/yd^2
Curasol	273	Synthetic Resin	3%	0.3 gal/yd^2
M166 & M1667	891	Latex	7%(M166) + 0.2%(M167)	0.5 gal/yd^2
Coherex CRF	847	Petroleum resin	25%	0.5 gal/yd^2
Sherman Process (no grass seed)	800	Straw mulch with emulsified asphalt	NA	NA
Sherman Process (with grass seed)	800	Straw mulch with emulsified asphalt	NA	NA
Terra Tack I	68	Vegetable Gum	0.3%	1.4 gal/yd^2

NA = not applicable.
* Percent compound in aqueous solution, unless otherwise indicated.

Control of Wind Erosion Dust Emissions From Storage Piles

This field study was designed to demonstrate the effectiveness of a windscreen, alone or in combination with other control measures, in reducing dust emissions due to wind erosion from an inactive waste storage pile. The windscreen was 8 feet high and 75 feet in length with a porosity of 50 percent. Control measures tested in combination with the windscreen were pile watering and chemical dust suppressants (Curasol and Genaqua). Concurrent upwind/downwind aerosol/windspeed measurements were made with real-time data retrieval around a storage pile protected with a windscreen.

The waste storage pile tested was not contaminated. It was assumed that the hazardous material would be integrally bound to the soil and therefore would be emitted at a rate directly proportional to the loss rate of soil from the pile. The soil selected was a very fine shredded topsoil. The desired size of the pile was determined to be large enough for accurate simulation of wind erosion action from a temporary waste storage pile, yet not so large that building the pile consumed a major portion of the study's resources. This "reasonable" size was estimated to be about 8 ft high with an elliptical base 25 ft by 20 ft, or about 100 tons of shredded topsoil.

All atmospheric particulate concentrations were measured with RAM-1 continuous aerosol monitors. These instruments emit a pulsed light across a continuous flowing sample airstream and sense the amount of light scattering with a silicon detector. The RAM-1's are stated to have an upper particle size range of 20-µm diameter, but are generally considered to have a 50 percent collection efficiency upper cut point of about 10 µm. Windspeed and wind direction were measured with a Met One system that included six windspeed sensors and one direction vane. The anemometer sensors had an accuracy of ±0.25 mph; the wind direction vane had an accuracy of ±5 degrees.

Both the RAM-1's and wind instruments were connected through a translator and an analog-to-digital convertor to an onsite Apple IIe computer, which collected the individual signals during testing; displayed the readings continuously; averaged them over 1-minute, 5-minute, and 1-hour time periods; and stored the averages for later retrieval. All the calibration values could be changed by inputting new values to the computer.

Both the upwind sampling stand and the main downwind stand had RAM-1's and windspeed sensors mounted at 3.1 and at 6.2 feet heights (Figure 1). The upwind stand also had a wind direction vane mounted at a 6.2 feet height. The third sampling stand, which was alternated between a location beside the main downwind stand and a distance farther downwind, had a RAM-1 and windspeed sensor only at a 3.1 feet height. A sixth windspeed sensor was located at the upwind edge of the pile at a height of 3.1 feet.

Chemical 1 was Curasol. It was first applied at a dilution of 4.0 percent at a rate of 0.6 gal/yd^2. On the next day, Curasol was applied at the same dilution at 1.0 gal/yd^2. Testing occured over a period of 0 to 3 days since application. Chemical 2 was Genaqua. It was applied at a dilution of 10 percent at a rate of 0.90 gal/yd^2. Testing was conducted over a period of 0 to 1 days since application. Watering occured at the rate of 0.9 gal/yd at the beginning of each test day. Testing occured at 0 to 6 hours since application.

Control of Dust Emissions From Active Site Cleanup

The operation selected for testing consisted of a front-end loader (FEL) and dump truck combination. The FEL scraped material from the surface, turned, traveled to the dump truck, and dumped its load into the truck. This activity simulates the most common method of loading contaminated soil into trucks for offsite disposal.

The exposure profiling method was used to sample the dust emissions downwind of the operation. This method employs a tower with multiple profiling heads to perform simultaneous multipoint isokinetic sampling over the cross section of the plume. The primary sampling instruments were profiling heads utilizing the stacked filter concept. The sampling head internally fractionates the dust sample by particle size. Three profiling towers were used. One tower was located upwind of the operation to measure the background concentration not attributable to source

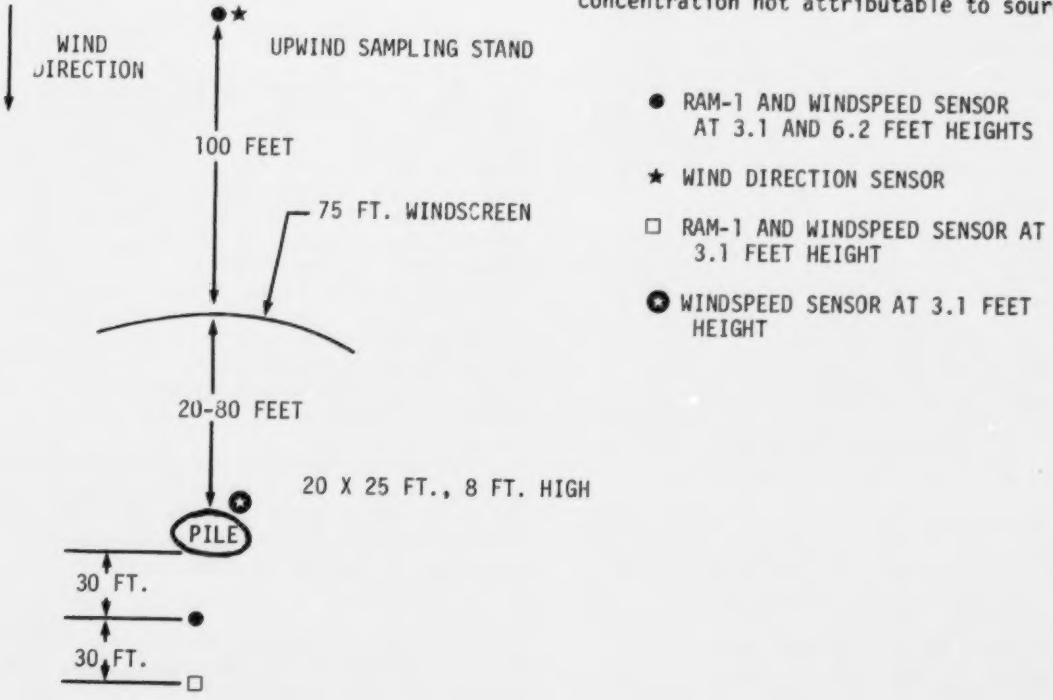
WIND DIRECTION
UPWIND SAMPLING STAND
100 FEET
75 FT. WINDSCREEN
20-80 FEET
20 X 25 FT., 8 FT. HIGH
PILE
30 FT.
30 FT.
● RAM-1 AND WINDSPEED SENSOR AT 3.1 AND 6.2 FEET HEIGHTS
★ WIND DIRECTION SENSOR
□ RAM-1 AND WINDSPEED SENSOR AT 3.1 FEET HEIGHT
✪ WINDSPEED SENSOR AT 3.1 FEET HEIGHT

Figure 1. Windscreen sampling configuration.

operation. A similar sampling tower was located downwind of the FEL scraping and traveling path to measure the scraping emissions. The other tower which was equipped with a horizontal crossarm, was located downwind of the dump truck to measure the dump cycle emissions. On each sampling day, both an uncontrolled and a controlled test were run. The control efficiency is merely the difference in emission rate per unit of activity between the uncontrolled and controlled tests.

Four control measures were tested. Control Measure 1 consisted of spraying the active working area of the FEL and dump truck with water. At those few hazardous waste sites where dust control measures are currently in use, this is the control being used. A portable 200-gallon tank, pump, generator, hose, and nozzle were used to spray the working area prior to the controlled test and again during the test to allow the field team to note drying of the surface and visible emissions. Watering amounts averaged 0.9 gal/yd^2 of surface area disturbed. More water was used on the travel paths of the FEL and dump truck than on the scraping area. These active travel areas dried much faster than the scraping area. It was noted throughout the testing that only the top 1 or 2 inches of the surface were dry. Below this dry crust, the soil was very damp.

For Control Measure 2, application procedures were identical to those used in the application of plain water; however, a surfactant (Johnson-Marsh Compound MR) was added to the water to form a 1:1000 dilution of surfactant to water. Somewhat less watering was needed for these controlled tests. Application of the water/surfactant mixture averaged about 0.75 gallon/yd^2 of surface area disturbed.

The concept behind Control Measure 3 was to create a flat spray or curtain that would totally cover the top of the truck box. The material is moistened as it passes through the spray. More importantly, the dust that is pushed upward from the bottom of the truck box by air turbulence is caught in the spray. A frame was built to support a line of six spray nozzles on each side of the length of the truck bed. The truck was then backed into position between the standards and the spray was turned on.

Because the spray heads were in constant operation throughout the test period, the amount of surfactant required averaged about 1.5 gal/yd^3 of material. In retrospect, the spray probably should have been operated only during the actual dump. This would have entailed the use of substantially less liquid. Although not tested, it is believed that control values would have been similar to those with the continuous spray.

For Control Measure 4, the spray system used for the water curtain was modified to produce a foam spray. Four foam-spray nozzles were used in this array, one at each corner of the truck bed. A mixture of water and surfactant, approximately 30:1 dilution, was used to generate a stream of foam. The system was only turned on during the dump. About 0.4 gal/yd^3 of mixture was used.

PROBLEMS ENCOUNTERED

Two problems were encountered with the exposed area study. Initially, vacuum surface samples of the test plots were taken in addition to the saltation catcher samples. The vacuum samples showed very high concentrations of the tracer, which led the investigators to believe that the vacuum negative pressure was lifting the dust-suppressant-formed crust. These data were not used. The second problem encountered was the rapid growth of vegetation on the initially bare plots. The plant stems destroy the crust, as explained in the results section.

The windscreen/storage pile emissions testing program went very well. Use of samplers with a particle size cut-off greater than 10 micrometers would have assisted in some of the data interpretation, however.

The active cleanup testing was continually hampered by wet weather, which kept conditions at the test site fairly moist. Additional engineering design refinements to both the surfactant and foam spray curtains probably would have resulted in improved control efficiencies.

RESULTS

Control of Wind Erosion Dust Emissions From Exposed Areas

Table 2 lists the results of the saltation sample analyses. Values over 75 ppm (the background concentration of zinc in the soil tested in each plot) indicate failure of the dust suppressant.

The data indicate integrity of the crust of all plots except Plots 5 and 8 on June 11 . Time since application varied from 17 to 31 days. Two weeks later, on June 25, only Plot 7 remained near the background level of zinc in the soil. Plot 11, to which the dust suppressant and tracer were applied later in the test period, was also effective after 30 days. No reapplications were tested; however, it is reasonable to assume that control after reapplication would be slightly greater than with the initial application.

A problem with all plots was the rapid growth of vegetation. All plots were stripped of vegetation before application of the tracer and dust-suppressant. The rationale for this action was that a dust suppressant spray could not form a dirt crust in the presence of vegetation stems, because the dust suppressant would not uniformly pass through the vegetation and reach the soil. When vegetation did grow, it penetrated the crust, and a small dirt pile was evident around each stem. Testing of these dust piles indicated large quantities of the tracer material (>200 ppm). When this finding is applied to hazardous waste sites, it is apparent that vegetation must be controlled if the crust is to stay intact. This can be done by spraying preemergent weed control on the plot before applying of the dust suppressant.

An alternate procedure to eliminating vegetative growth would be to encourage it. Products are available that are temporary soil binders impregnated with grass seed. While grass was beginning to grow, the same weed problem just described would occur. Assuming a thick stand of grass did grow, control would probably not be 100 percent, as some loose dirt would always be present between grass stems. Chemical dust suppressants sprayed on thick grass stands would not be effective because the suppressant would stick to the stems and little would penetrate to the soil.

Control of Wind Erosion Dust Emissions From Storage Piles

A total of 82 tests were taken--47 with the pile surface dry, 14 with watering, and 21 with chemicals applied. Hourly average windspeeds varied from 4.1 to 24.1 mph. The distance of the screen from the pile dictated the amount of windspeed reduction observed at the pile:

TABLE 2. SELECTED SALTATION SAMPLER RESULTS (ppm)

Test plot	Date established	Sample date (1984) 6-11	6-25	7-13	7-30	8-22	8-31
1	5/24/84	78	97	111	155	183	170
2	5/11/84	71	114	121	322	172	204
3	5/18/84	42	163	125	<65	252	120
4	5/25/84	69	105	116	166	140	119
5	5/18/84	91	172	152	58	141	217
6	5/24/84	75	111	130	97	167	128
7	5/24/84	64	76	96	152	160	122
8	5/24/84	113	162	100	118	206	214
9	7/10/84	NS	NS	NS	NS	513	132
10	7/10/84	NS	NS	NS	NS	157	103
11	7/31/84	NS	NS	NS	NS	48	72

NS = no samples.

Screen-to-pile dist., ft.	Av. windspeed, % of incoming
20 (2.5 screen heights)	30
50 (6.3 screen heights)	50
80 (10 screen heights)	85

These reductions were fairly independent of incoming windspeed. They were also consistent with reductions reported previously in the literature.

The screen did reduce windspeeds by the amount anticipated, but this did not result in commensurate reductions in particulate concentrations coming from the pile. Without the windscreen in place, net concentrations from the pile ranged from 2 to 34 μg/m³, averaging 18 μg/m³, at the 6.2 foot height. Reductions in net particulate concentrations attributed to the windscreen are shown in Table 3. On the dry pile, the screen was able to reduce wind erosion slightly if it was placed close to the pile. The 2 meter samplers at 50 or 80 ft. distance appeared to be out of the area sheltered by the screen, and possibly in an area of windscreen induced wind shear.

Taken in total, the results indicate that the windscreen did not produce consistent reductions in wind erosion. An explanation for the windscreen's performance, developed after review of all the data from this study, is that wind erosion emission rates in the less than 10 micrometer size range are fairly constant at windspeeds above the threshold of about 7 mph (hourly average). The additional emissions associated with high wind erosion losses at high windspeeds are larger particles that are not detected by the RAM-1's. The windscreen may control larger particles more effectively, but these particles are less of a threat because they exceed the inhalable size and are not carried long distances in the wind.

In tests with different pile-to-windscreen distances, it was found that a distance of 2.5 screen heights was superior to a distance of 6.3 to 10.0 screen heights.

Chemical dust suppressants without the windscreen were very effective in reducing wind erosion over the four days since the application was tested. The tests of the windscreen in combination with watering or chemical dust suppressants inexplicably showed that the screen was counterproductive.

In terms of dust control for an inactive hazardous waste storage pile, this study showed that chemical dust suppressants provided the greatest and most consistent reduction in wind erosion. No firm evidence was uncovered to indicate that windscreens provided any incremental control beyond that of the chemicals when the two controls were applied together.

Based on requirements for putting the controls in place and their time of effectiveness, experience in this study also showed that chemical dust suppressants were preferable to either windscreens or watering, even if the piles must be restabilized on a daily or weekly basis because of intermittent activity. One person using simple equipment could spray

TABLE 3. REDUCTION IN NET PARTICULATE CONCENTRATIONS ATTRIBUTED TO A WINDSCREEN

Screen dist., ft.	Surface condition	Est. reduction due to screen, μg/m³ @ 3.1 ft. ht.	@ 6.2 ft. ht.
20	dry	3 to 4	0 to 5
50 or 80	dry	3 to 4	-8 to -10
20 or 50	watered	negative	negative
20 or 50	chemicals applied	-2	-3 to -6

the pile with chemicals in 20 minutes or less. The windscreen took two persons several hours for initial installation and two persons 30 minutes to move the screen when the wind shifted. A shift in wind direction would require no change to the chemically stabilized pile. No information was obtained during this study on the durability of the chemical crust, except that chemical 1 was on the pile for 4 days with no apparent reduction in effectiveness. Watering was effective for only a few hours.

This field study showed that the use of windscreens probably would only be worthwhile for dust control from an inactive pile in situations where chemicals could not completely stabilize the soil in the pile and drifting of large particles posed a threat of offsite contamination.

Control of Dust Emissions from Active Site Cleanup

Water spraying over the area being worked by the FEL and truck resulted in a control efficiency of 42 percent for <30-micrometer particles (TSP) and 64 percent for <2.5-micrometer particles (FP). (See Table 4.) TSP emissions from the dump cycle were reduced by 63 percent and FP emissions by 70 percent. Adding surfactant to the water increased TSP control efficiencies slightly and permitted a reduction in the quantity of water used. The control efficiency for TSP generated by FEL travel/scraping increased from 42 to 63 percent with the addition of the surfactant. Other control values had smaller increases.

Both of the spray-curtain control measures were shown to be less effective than the area spray with the water/surfactant mixture; however, a redesign of the controls used may produce higher efficiencies. Of the two, the water curtain was somewhat better for control of dust from the dump cycle than the foam curtain. If one of these controls were used in conjunction with the water/surfactant area spray, the resulting control efficiency would probably be significantly greater than for either one alone.

Drier conditions than were experienced during testing would require greater quantities of water. Because the goal of 100 percent control efficiency is unlikely with these technologies, the potential for subsequent human exposure impacts still exists.

Use of Chemical Dust Suppressants

In general, chemical dust suppressants are often by-products or waste materials. Some dust suppressants have been found to contain heavy metals and PCB's. Before applying a dust suppressant, the user should analyze the dust suppressant for toxic substances and test for reactivity between the dust suppressant and contaminated soil.

TABLE 4. SUMMARY OF RESULTS OF ACTIVE CLEANUP TESTING

Operation	Control measure	Control efficiency, % <2.5 μm	<30 μm
FEL travel and scraping	Area spray-water (0.9 gal/yd²)	64	42
	Area spray-water/surfactant (0.75 gal/yd²)	70	63
FEL dump	Area spray-water (0.9 gal/yd²)	66	69
	Area spray-water/surfactant (0.9 gal/yd²)	62	77
	Water curtain (1.5 gal/yd³)	56	50
	Foam curtain (0.4 gal/yd³)	41	46

PRACTICAL METHODS FOR DECONTAMINATING BUILDINGS, STRUCTURES, AND EQUIPMENT AT SUPERFUND SITES

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ABSTRACT

Practical methods for decontaminating buildings, structures, and equipment at Superfund sites are described in a new EPA handbook. Techniques presented include: asbestos abatement, absorption, demolition, dismantling, dusting/ vacuuming/wiping, encapsulation/ enclosure, gritblasting, hydroblasting/waterwashing, painting/coating, K-20, scarification, RadKleen, solvent washing, steam cleaning, vapor-phase solvent extraction, acid etching, bleaching, flaming, drilling and spalling, microbial degradation, and photochemical degradation. Method descriptions include a general discussion of the procedure, advantages, disadvantages, applicability, effectiveness, engineering considerations (including building preparation, process description, and equipment needs), safety requirements, waste disposal, and costs. Potential decontamination techniques for a given contaminant/structural material combination are identified in a matrix. Other information presented in the manual includes stepwise guidance for developing a cost-effective decontamination strategy, case studies illustrating the actual application of many decontamination methods, cost analyses for the application of each method to a model building, a discussion of worker health and safety precautions, and a summary of available sampling techniques.

INTRODUCTION

Cleanup of abandoned hazardous waste sites is the Nation's top environmental priority in the 1980's. In a Superfund reauthorization bill before the House (HR 5640), a mandatory cleanup schedule is being considered that would require EPA to list 1600 sites on the National Priorities List by January 1, 1988, and to begin remedial investigations and feasibility studies at each site within 6 months of the date it is listed. This greater level of activity will increase the need for decontamination of buildings, structures, and equipment.

In response to this growing need, the U.S. EPA funded this study to develop a general decontamination guide or handbook for government personnel, cleanup contractors, and other individuals responsible for planning and executing decontamination activities at Superfund sites.

As an initial step in the development of the handbook, a survey of building and equipment decontamination practices at Superfund sites was conducted. This survey revealed that the methods used to remove contaminants from buildings, structures, and equipment are few in number and rarely documented in detail. For example, it is common practice to steam clean equipment such as backhoes, bulldozers, and drilling augers, but testing to verify that the contaminants of concern have been adequately removed is generally not performed. Contaminated buildings and structures are seldom cleaned and returned to active use. More often, they are closed and barricaded to prevent further entry and exposure, or they are torn down and buried in secure landfills. Contaminated underground structures such as tanks, sumps, and sewers are sometimes filled in place with concrete to prevent their reuse. These findings clearly pointed to the need for basic guidance material dealing with the identification and selection of appropriate decontamination methods, as well as their application to various contaminants and structural materials.

DECONTAMINATION METHODS AND THEIR APPLICATION

The user's manual which was developed contains information on several methods that have potential use for decontaminating buildings, structures, and equipment. The methods presented range from classical techniques, which have been extended in application to the cleanup of hazardous waste sites, to developmental concepts, which have shown strong potential as decontamination techniques in laboratory and initial field demonstrations. Method descriptions include a general discussion of the procedure, advantages, disadvantages, applicability, effectiveness, engineering considerations (including building preparation, process description, and equipment needs), safety requirements, waste disposal, and costs. The following paragraphs briefly describe each example method:

Asbestos abatement consists of four techniques: removal, encapsulation, enclosure, and special operations. In removal operations, all friable asbestos-containing building materials are completely removed to eliminate the release of asbestos fibers into the air. The other techniques leave the asbestos fibers in place but limit potential exposure levels through various treatment, maintenance, and inspection procedures.

Absorption is widely used in industrial settings to clean up chemical and other liquid spills. It is also commonly used by emergency response teams such as fire departments to absorb accidental spills on highways and other surfaces. This method is most applicable immediately following liquid contaminant spills. Spills rapidly penetrate most surfaces, and absorbents act to contain contaminants and prevent such penetration. Depending on the surface and time elapsed since the spill, further decontamination procedures may have to be employed.

Demolition refers to the total destruction of a building, structure, or piece of equipment. Specific demolition techniques include complete burndown, controlled blasting, wrecking with balls or backhoe-mounted rams, rock splitting, sawing, drilling, and crushing. Many of these techniques have been employed for nuclear facility decontamination and for the clean up of military arsenals.

Dismantling refers to the physical removal of selected structures (such as contaminated pipes, tanks, and other process equipment) from buildings or other areas. Dismantling can be the sole activity of decontamination efforts (e.g., removal of contaminated structures from an otherwise clean building), or it can be used in the initial stage of a more complex building decontamination effort (e.g., removal of structures prior to flaming, hydroblasting, or other cleanup techniques).

Dusting/vacuuming/wiping is simply the physical removal of hazardous dust and particles from building and equipment surfaces by common cleaning techniques. Variations include vacuuming with a commercial or industrial-type vacuum; dusting off surfaces such as ledges, sills, pipes, etc., with a moist cloth or wipe; and brushing or sweeping up hazardous debris. Dusting and vacuuming are applicable to all types of particulate contaminants, including dioxin, lead, PCB's, and asbestos fibers, and to all types of surfaces. Dusting/vacuuming/wiping is the state-of-the-art method for removing dioxin-contaminated dust from the interior of homes and buildings.

Encapsulation/enclosure physically separates contaminants or contaminated structures from building occupants and the ambient environment by means of a barrier. An encapsulating or enclosing physical barrier may take different forms; among them are plaster, epoxy, and concrete casts and walls. Acting as an impenetrable shield, a barrier keeps contaminants inside and away from clean areas, thereby alleviating the hazard. As a result, contamination of part of a structure will not result in the contamination of adjacent areas. Encapsulation has been used on damaged asbestos insulation, leaky PCB-contaminated electrical transformers, and open maintenance pits and sumps contaminated by heavy metals.

Gritblasting is a surface removal technique in which abrasive materials (such as sand, alumina, steel pellets, or glass beads) are used for uniform removal of contaminated surface layers from a building or structure. Gritblasting has been used since 1870 to remove surface layers from metallic and ceramic surfaces, and is currently used extensively throughout industry. For example, sandblasting is commonly used to clean the surfaces of old brick and stone buildings. The gritblasting method is applicable to all surface contaminants except some highly sensitive explosive contaminants such as lead azide and lead styphnate. This method is applicable to all surface materials except glass, transite, and Plexiglas.

Hydroblasting/waterwashing refers to the use of a high-pressure (500 to 50,000 psi) water jet to remove contaminated debris from surfaces. The debris and water are then collected and thermally, physically, or chemically decontaminated. Hydroblasting has been used to remove explosives from projectiles, to decontaminate military vehicles, and to decontaminate nuclear facilities. Hydroblasting also has been employed commercially to clean bridges, buildings, heavy machinery, highways, ships, metal coatings, railroad cars, heat exchanger tubes, reactors, piping, etc. Off-the-shelf equipment is available from many manufacturers and distributors.

Painting/coating includes three types of decontamination techniques: 1) the removal of old layers of paint containing high levels of toxic metals such as lead; 2) the use of fixative/stabilizer paint coatings; and (3) the use of adhesive-backed strippable coatings. In the first technique, paint containing lead in excess of 0.06 percent is removed from building surfaces by commercially available paint removers and/or physical means (scraping, scrubbing, waterwashing). Resurfacing or further decontamination efforts may be necessary. The second technique involves the use of various agents as coatings on contaminated surfaces to fix or stabilize the contaminant in place, thereby decreasing or eliminating exposure hazards. Potentially useful stabilizing agents include molten and solid waxes, carbowaxes (polyoxyethylene glycol), saligenin (α,2-dihydroxy toluene), organic dyes, epoxy paint films, and polyester resins (also see K-20). The stabilized contaminants can be left in place or removed later by a secondary treatment. In some cases, the stabilizer/fixative coating is applied in situ to desensitize a contaminant (such as an explosive residue) and prevent reaction or ignition during some other phase of the decontamination process (for example, to prevent explosions during dismantling or demolition). In the third technique, the contaminated surface is coated with a polymeric mixture. As the coating polymerizes, the contaminant becomes entrained in the lattice or attached to the polymer molecules. As the polymer layer is peeled off, the residue is removed with it. It may be possible, in some cases, to add chemicals to the mixture to inactivate the contaminants.

K-20 coating is a newly developed commercial product sold as a building decontaminant. It acts as a penetrant and sealer that basically encapsulates and immobilizes toxic contaminants in place. Its effectiveness is not fully known. Although it acts more like a barrier than a detoxifier, early laboratory evidence indicates K-20 may facilitate chemical degradation as well as physical separation of some contaminants.

Scarification is a method that can be used to remove up to an inch of surface material from contaminated concrete or similar materials. The scarifier tool consists of pneumatically operated piston heads that strike the surface, causing concrete to chip off. This technique has been used in the decommissioning of nuclear facilities and in the cleanup of military arsenals.

RadKleen is the trademark for a commercial process involving the use of Freon 113 (1,1,2-trichloro-1,2,2-trifluoroethane or $C_2F_3Cl_3$) for solvent extraction of soluble contaminants from various materials. The solvent is pressure-sprayed onto the surface, then collected, treated, and recycled. The RadKleen process is currently used for cleaning radioactive material from various surfaces. It has been applied to chemical agents on small objects, and thus field capability has been demonstrated. Studies have been conducted for agent-contaminated clothing materials, such as polyester-cotton, Nomex, butyl rubber gloves, and charcoal-impregnated cloth. Although this method has not been demonstrated for removing contaminants from building surfaces, it looks very promising.

Solvent washing refers to the application of an organic solvent (e.g., acetone) to the surface of a building to solubilize contaminants. This technique has not yet achieved widespread use in building decontamination, although it is beginning to be used in the decommissioning of nuclear facilities. The method needs further development in application, recovery, collection, and efficiency. The hot solvent soaking process has been shown to be effective in decontamination of PCB-contaminated transformers.

Steam cleaning physically extracts contaminants from building walls and floors, and from equipment. The steam is applied through hand-held wands or automated systems, and the condensate is collected in a sump or containment area for treatment. This method is currently used by explosives handling and manufacturing facilities. It has also been used to remove dioxin-contaminated soil from vehicles and drilling equipment in Times Beach, Missouri.

Vapor-phase solvent extraction is a method in which an organic solvent with a relatively low boiling point (such as methyl chloride or acetone) is heated to vaporization and allowed to circulate in a contaminated piece of equipment or enclosed area. The vapors permeate the contaminated materials, where they condense, solubilize contaminants, and diffuse outward. The contaminant-laden liquid solvent is collected in a sump and treated to allow recycling of the solvent. This method has not yet been applied to building decontamination, although it is believed to have good potential.

Acid etching of a contaminated surface is used to promote corrosion and removal of the surface layer. Muriatic acid (hydrochloric acid) is used to remove dirt and grime from brick building surfaces in urban areas and to clean metal parts (e.g., pickle liquors from metal finishing operations). The resulting contaminated debris is then neutralized and disposed of. Thermal or chemical treatment of the removed material may be required to destroy the contaminant before disposal. Although this technique is not known to have been applied to chemically contaminated building surfaces, it is believed to have good potential.

Bleaching formulations are applied to a contaminated surface, allowed to react with contaminants, and removed. Application usually occurs in conjunction with other decontamination efforts, such as the use of absorbents and/or water-washing. Bleach has been used as a decontaminant against mustard, G and V chemical agents, and (experimentally) organophosphorous pesticides.

Flaming refers to the application of controlled high temperature flames to contaminated noncombustible surfaces, providing complete and rapid destruction of all residues contacted. The flaming process has been used by the Army to destroy explosive and low-level radioactive contaminants on building surfaces. Its applicability to other contaminants is not well known. This surface decontamination technique can be used on painted and unpainted concrete, cement, brick, and metals. Subsurface decontamination of building materials may be possible, but extensive damage to the material would probably result. This technique can involve high fuel costs.

Drilling and spalling can remove up to 2 inches of contaminated surface material from concrete or similar materials. This technique consists of drilling holes (1 to 1-1/2 in. diameter) approximately 3 inches deep. The spalling tool bit is inserted into the hole and hydraulically spreads to spall off the contaminated concrete. The technique can achieve deeper penetration (removal) of surfaces than other surface-removal techniques and it is good for large-scale applications. However, the treated surface is very rough and coarse and may require resurfacing (i.e., capping with concrete). The drilling and spalling method has been used in the decommissioning of nuclear facilities.

Microbial degradation is a developing process whereby contaminants are biologically decomposed by microbes capable of utilizing the contaminant as a nutrient source. Conceptually, microbes are applied to the contaminated area in an aqueous medium and allowed to digest the contaminant over time; the microbes are then destroyed chemically or thermally, and washed away. Microbial degradation as a building decontamination technique is not well developed.

Photochemical degradation refers to the process of applying intense ultraviolet light to a contaminated surface for some period of time. Photodegradation of the contaminant follows. In recent years, attention has been focused on this method because of its usefulness in degrading chlorinated dioxins (TCDD in particular). Three conditions have been found to be essential for the process to proceed: 1) the ability of the compound to absorb light energy; 2) the availability of light at appropriate wavelengths and intensity, and 3) the presence of a hydrogen donor.

Potential decontamination techniques for a given contaminant/structural material combination may be identified with the aid of a matrix (see Table 1). Each cell in the matrix represents a specific contaminant/surface combination and contains numbers corresponding to decontamination methods just described that either have been used in the specific interactions or have the potential for such use. Each method may be used alone or possibly in conjunction with one or more of the other procedures. The reader is cautioned to remember that although these decontamination methods have been or could be used for the mix of the conditions described in the matrix, the effectiveness of the application in many cases is not known or has not been adequately documented or verified. Therefore, it is recommended that the method descriptions presented in the manual be used to evaluate each potential method on a site-specific basis for efficiency, wastes generated, equipment and support facilities needed, time and safety requirements, structural effects, and costs. Also, pilot-scale testing of each method (or combination of methods) should be performed prior to full-scale implementation to determine the effectiveness of the strategy.

OTHER INFORMATION

In addition to the method descriptions, the manual presents guidelines for developing a successful decontamination strategy, as outlined in Figure 1. Case studies illustrating the actual application of many of the decontamination methods described earlier are also included in the manual. Table 2 shows a summary of these case studies, indicating the contaminants present and the decontamination methods used in each case. Finally, the manual includes cost analyses for the application of each method to a model building, a discussion of worker health and safety precautions, and information on available sampling methods.

CONCLUSIONS

The handbook provides much of the guidance needed by Superfund site cleanup personnel for decontaminating buildings, structures, and equipment. However, many of the decontamination methods currently in use have not been verified and documented with respect to their effectiveness on various contaminant/material combinations. For example, the degree to which steam cleaning removes dioxin-contaminated soil particles from drilling augers has not been established. Additional research to demonstrate the effectiveness of available methods under various conditions is badly needed. Also, decontamination methods that have not previously been applied to specific contaminant/substrate combinations, but show a strong potential applicability, should be validated in pilot investigations. Additions/deletions to the matrix should be made accordingly. New decontamination technologies that become available also should be evaluated and added to the matrix.

TABLE 1. EXAMPLES OF PRACTICAL DECONTAMINATION METHODS FOR VARIOUS CONTAMINANTS AND STRUCTURAL MATERIALS[a]

Contaminant	Material						Equipment and auxiliary structures
	Brick	Concrete	Glass	Metal	Plastic	Wood	
Asbestos	1,3,4,5,6	1,3,5,6	1,3,4,5,6	1,3,4,5,6	1,3,4,5,6	1,3,4,5,6	1,3,4,5,6
Acids	2,3,4,6,7,8, 9,13	2,3,6,7,8,9, 10,13,18	2,3,4,6,9,13	2,3,4,6,7,8, 9,13	2,3,4,6,9,13	2,3,4,6,7,9, 13	2,3,4,6,7,8, 9,13
Alkalis	2,3,4,6,7,8, 9,13,15	2,3,6,7,8,9, 10,13,15,18	2,3,4,6,9,13 15	2,3,4,6,7,8, 9,13,15	2,3,4,6,9,13	2,3,4,6,7,9, 13,15	2,3,4,6,7,8, 9,13,15
Dioxins	2,3,4,5,6,7, 9,11,12,13, 14,19,20,21	2,3,5,6,7,9, 10,11,12,13, 14,18,19,20, 21	2,3,4,5,6,9, 11,12,13,14, 20,21	2,3,4,5,6,9, 11,12,13,14, 20,21	2,3,4,5,6,9, 11,12,13,14, 20,21	2,3,4,5,6,9, 11,12,13,14, 19,20,21	2,3,4,5,6,7, 8,9,11,12,13, 14,20,21
Explosives	2,3,4,6,7,8, 9,11,12,13, 14,17	2,3,6,7,8,9, 10,11,12,13, 14,17,18	2,3,4,6,11, 12,13,14	2,3,4,6,7,8, 9,11,12,13, 14,17	2,3,4,6,9,11, 12,13,14	2,3,4,6,7,9, 11,12,13,14	2,3,4,6,7,8, 9,11,12,13, 14,17
Heavy metals and cyanide	2,3,4,5,6,7, 8,9,12,13,14, 15,19	2,3,5,6,7,8, 9,10,12,13, 14,15,18,19	2,3,4,5,6,12, 13,14,15	2,3,4,5,6,7, 8,9,12,13,14, 15	2,3,4,5,6,9, 12,13,14	2,3,4,5,6,7, 9,12,13,14, 15,19	2,3,4,5,6,7, 8,9,12,13,14, 15
Low-level radiation	2,3,4,6,7,8, 9,11,12,13, 14,17	2,3,6,7,8,9, 10,11,12,13, 14,17,18	2,3,4,5,6,11, 12,13,14	2,3,4,6,7,8, 9,11,12,13, 14,17	2,3,4,6,9, 11,12,13,14	2,3,4,6,7,9, 11,12,13,14	2,3,4,6,7,8, 9,11,12,13, 14,17
Organic solvents	2,3,4,6,7,8, 9,11,12,13, 14,20,21	2,3,6,7,8,9, 10,11,12,13, 14,18,20,21	2,3,4,6,9,11, 12,13,14,20, 21	2,3,4,6,7,8, 9,11,12,13, 14,20,21	2,3,4,6,9,11, 12,13,14,20, 21	2,3,4,6,7,9, 11,12,13,14, 20,21	2,3,4,6,7,8, 9,11,12,13, 14,20,21
Pesticides	2,3,4,5,6,7, 8,9,11,12,13, 14,16,19,20, 21	2,3,5,6,7,8, 9,10,11,12, 13,14,16,18, 19,20,21	2,3,4,5,6,9, 11,12,13,14, 20,21	2,3,4,5,6,7, 8,9,11,12,13, 14,16,20,21	2,3,4,5,6,9, 11,12,13,14, 20,21	2,3,4,5,6,7, 9,11,12,13, 14,16,19,20, 21	2,3,4,5,6,7, 8,9,11,12,13, 14,16,20,21
PCB's	2,3,4,5,6,7, 8,9,11,12,13, 14,19,20,21	2,3,5,6,7,8, 9,10,11,12, 13,14,18,19, 20,21	2,3,4,5,6,11, 12,13,14,20, 21	2,3,4,5,6,7, 8,9,11,12,13, 14,20,21	2,3,4,5,6,9, 11,12,13,14, 20,21	2,3,4,5,6,7, 9,11,12,13, 14,19,20,21	2,3,4,5,6,7, 8,9,11,12,13, 14,20,21

Key for decontamination methods:

1. Asbestos abatement
2. Absorption[b]
3. Demolition[c]
4. Dismantling
5. Dusting/vacuuming/wiping[d]
6. Encapsulation
7. Gritblasting[e]
8. Hydroblasting/waterwashing
9. Painting/coating
10. Scarification[e,f]
11. RadKleen
12. Solvent washing
13. Steam cleaning
14. Vapor-phase solvent extraction
15. Acid etching
16. Bleaching
17. Flaming
18. Drilling and spalling[e,f]
19. K-20 coating
20. Microbial degradation
21. Photochemical degradation

[a] Refer to individual method descriptions in Section 4 to determine whether an indicated technique has actually been used to treat a particular contaminant/structural material combination, or whether the method is viewed as potentially applicable.

[b] Applicable only to liquids.

[c] Some contaminant residues (e.g., asbestos, explosives, toxic residues) may have to be neutralized, stabilized, or removed prior to demolition to prevent explosions or emissions.

[d] Applicable only to particulates and solids.

[e] Not recommended for removing highly toxic residues or highly sensitive explosives, unless particulates can be controlled.

[f] Applicable only to concrete.

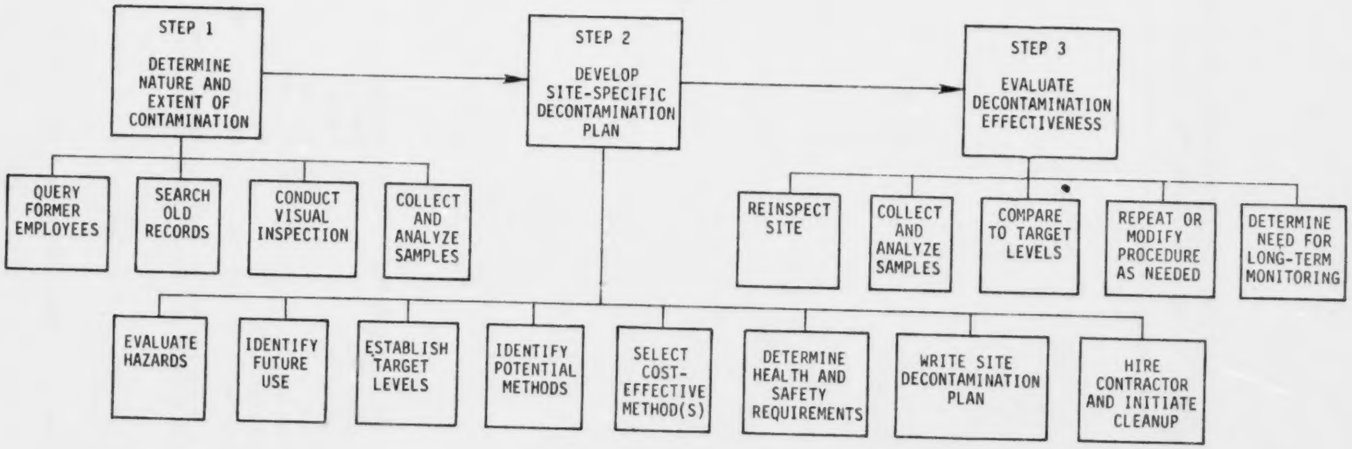
STEP 1
DETERMINE NATURE AND EXTENT OF CONTAMINATION
STEP 2
DEVELOP SITE-SPECIFIC DECONTAMINATION PLAN
STEP 3
EVALUATE DECONTAMINATION EFFECTIVENESS
QUERY FORMER EMPLOYEES
SEARCH OLD RECORDS
CONDUCT VISUAL INSPECTION
COLLECT AND ANALYZE SAMPLES
REINSPECT SITE
COLLECT AND ANALYZE SAMPLES
COMPARE TO TARGET LEVELS
REPEAT OR MODIFY PROCEDURE AS NEEDED
DETERMINE NEED FOR LONG-TERM MONITORING
EVALUATE HAZARDS
IDENTIFY FUTURE USE
ESTABLISH TARGET LEVELS
IDENTIFY POTENTIAL METHODS
SELECT COST-EFFECTIVE METHOD(S)
DETERMINE HEALTH AND SAFETY REQUIREMENTS
WRITE SITE DECONTAMINATION PLAN
HIRE CONTRACTOR AND INITIATE CLEANUP

Figure 1. Flow diagram illustrating sequence of steps for developing a decontamination strategy.

TABLE 2. SUMMARY OF CASE STUDIES

Site	Contaminants present	Decontamination methods
Homes and other buildings Seveso, Italy	TCDD	Dusting/vacuuming/wiping Painting/coating Dismantling Demolition
State Office Building Binghamton, New York	PCB's, TCDD, TCDF	Dusting/vacuuming/wiping Dismantling
Sontag Road area[a] St. Louis County, Missouri	TCDD	Dusting/vacuuming/wiping Insulation removal Scrubbing (equipment only) Steam cleaning (equipment only)
One Market Plaza Office Complex San Francisco, California	PCB's, PCDD, PCDF	Insulation removal Dusting/vacuuming/wiping Solvent washing Scraping Painting/coating K-20 Gritblasting Scarification/jackhammering Dismantling Hydroblasting/waterwashing (equipment only)
Frankford Arsenal Philadelphia, Pennsylvania	Explosives	Flaming Demolition
	Asbestos	Asbestos removal
	Radiological residues	Dusting/vacuuming/wiping Hydroblasting/waterwashing Scarification Gritblasting Dismantling
	Heavy metals	Painting/coating
Office building New England	Asbestos	Asbestos encapsulation
Luminous Processes, Inc.[a] Athens, Georgia	Low-level radiation	Paint stripping/sanding Hydroblasting/waterwashing Dismantling
Chemical Metals Industries, Inc.[a] Baltimore, Maryland	Heavy metals, acids, alkalis, cyanide- and ammonia-bearing compounds, salts, and solids and sludges of unknown composition	Gritblasting Dismantling

[a] Superfund site.

ACKNOWLEDGMENTS

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IN SITU TREATMENT OF CONTAMINATED GROUNDWATER AND SOILS, KELLY AIR FORCE BASE, TEXAS

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ABSTRACT

In situ biological degradation has been selected for field demonstration at a waste disposal site at Kelly Air Force Base, Texas. The in situ biodegradation treatment of this site will involve the introduction of oxygen and nutrients to the subsurface and the circulation of groundwater by an extraction/infiltration system. Extensive investigation has been necessary to characterize the site, demonstrate the effectiveness of the biological treatment in the laboratory, and determine the modifications of site conditions required to approach the optimum for treatment. The four elements of the project necessary to provide the information required for the design and implementation of the field demonstration are 1) geological investigation and site characterization, 2) determination of the contaminant profile, 3) microbiological investigation, and 4) laboratory biodegradation investigation. The results of each element of the project are summarized and recommendations for design and implementation of the field demonstration are made based on these results.

INTRODUCTION

In situ biodegradation of contaminated soil and groundwater is based on the concept of stimulating the indigenous subsurface microbial population to degrade organic contaminants. To enhance biodegradation it is necessary to "correct" conditions that may limit bacterial growth and metabolism. In the subsurface environment oxygen is usually the limiting condition for aerobic biodegradation, although inorganic nutrients such as nitrogen, phosphorus and trace metals, and sometimes organic carbon, can also be limiting. Optimum conditions can be provided by injecting an oxygen source and nutrients into the subsurface as required, and circulating the groundwater using an injection system and extraction wells.

The in situ approach has some distinct advantages over conventional "pump and treat" technologies which involve pumping the water to the surface for treatment. First, because the active treatment zone is in the subsurface, in situ biodegradation has the potential to remove contaminants sorbed to the soil matrix. Second, in situ biodegradation requires minimal equipment, i.e., that associated with pumping, nutrient mixing and injection, and oxygen addition. "Pump and treat" technologies are generally more expensive because of the cost of an aboveground treatment system.

The most common problem encountered in previous in situ biodegradation work has been the limited amount of oxygen that can be supplied by aeration to the subsurface microorganisms (3, 4). Hydrogen peroxide (H_2O_2) is now being investigated as an alternative oxygen source. Although hydrogen peroxide is cytotoxic at higher concentrations, studies indicate that it can be added to groundwater at concentrations up to 100 ppm without adverse effects assuming proper formulation and application control (5). This amount provides five times more oxygen than can be introduced by aeration alone.

The Air Force has selected a waste disposal site at Kelly Air Force Base, Texas, for the field demonstration of an in situ technology. The site was originally used as a disposal pit for chromium sludges and other electroplating wastes. Prior to being closed in 1966, the site had been used as a chemical evaporation pit for chlorinated solvents, cresols, chlorobenzenes, and other chemicals.

JRB has chosen in situ biological degradation as most suitable for demonstration at the Kelly site. The site offers a number of advantages with respect to conducting an in situ biodegradation demonstration project, namely small size, near optimum soil and groundwater temperatures, the presence of biodegradable organics, presence of a perched aquifer not used for drinking, and a highly adaptive and substantial microbial population. The in situ treatment will involve providing nutrients, oxygen in the form of hydrogen peroxide to the subsurface, and circulating water by use of an extraction/infiltration system.

PURPOSE

The purpose of this project is to evaluate the feasibility and effectiveness of in situ biological degradation as a hazardous waste disposal site remediation technique. Field demonstrations of in situ biodegradation using hydrogen peroxide as an oxygen source are being conducted. However, all in situ biodegradation projects involving the use of hydrogen peroxide have been associated with gasoline leaks or spills of one or a relatively few chemicals. No information is available on the feasibility of using hydrogen peroxide as an oxygen source to enhance biological degradation in the presence of the complex variety of contaminants which can be present at hazardous waste disposal sites. Indeed, little work has been done to evaluate the effectiveness of in situ biodegradation at hazardous waste disposal sites.

APPROACH

This project has included a feasibility study and a laboratory study to evaluate the potential in situ treatment options and optimize their design. This is being followed by the design, installation, startup, and monitoring of the field demonstration of an in situ treatment system.

The investigative work which has been required for the design and implementation of the field demonstration has involved the following four elements:

- Geological investigation and site characterization
- Determination of the extent and nature of contamination
- Microbiological investigation
- Laboratory biodegradation research.

The geological investigation required to characterize the site has included the determination of site stratigraphy and site hydrology, which is being completed, and permeability studies which are currently being conducted by K.W. Brown Associates.

Priority pollutant analysis according to EPA recommended protocol for organics and inorganics has been completed for five borehole and three groundwater locations. Further sampling and analysis may be required based on requirements for design of the demonstration system.

Microbiological investigation carried out at Memphis State University involved enumerating indigenous microorganisms using a direct-count procedure, acridine orange epiflourescence, and a viable agar plate-counting procedure using seven different media. These enumerations were conducted to determine if viable bacterial populations existed at the site, and if there was any correlation of bacterial number with soil type, depth, location, and contaminant concentrations.

The laboratory biodegradation study is nearing completion. This study is necessary to determine, under given treatment conditions, what will biodegrade, the rates of biodegradation, the intermediate metabolites and metabolic products, and any problems that may be encountered with a particular treatment approach. The study has involved GC/MS analyses of 240 ml closed microcosms consisting of soil/groundwater slurries. Microcosms are being sacrificed for analysis on days 0, 25, 50, and 100. Aerobic and anaerobic biodegradation is being evaluated according to the following amendment regime*

- aerobic: H_2O_2 + nutrients
- aerobic control: sterilized H_2O_2 + nutrients
- aerobic: oxygen + nutrients
- anaerobic + nutrients
- anaerobic control: sterilized anaerobic + nutrients.

RESULTS

Geological Characterization

The characterization of the site has been complicated by the stratigraphic heterogeneity of the on-site soils. The high degree of variability, including gravel lenses and layers, has made it very difficult to construct subsurface profiles. It also compounds the difficulty of predicting the flow of contaminants in the subsurface, thus complicating the approach for treatment of the site.

The site under study is an elevated peninsula-like landform, surrounded on three sides by surface drainage (Figure 1). Leon Creek borders the site on the west, and an unnamed surface drainage ditch borders it on the east and south. The site is approximately 15 to 20 feet above this drainage, and has steeply sloping sides. It will be difficult to isolate the site from surface waters and to prevent contaminated groundwater from exiting the sides of the site. Only the very dry weather conditions prevent such losses at this time.

There is an abundance of fine-grained soils and sludges at this site. Although no hard data are yet available, the hydraulic conductivity of these materials can be expected to be low. This means that the task of delivering treatment solutions to the contaminated areas is likely to be difficult and lengthy.

The problems with particle size also extend to coarser materials that are present. The gravel layers and lenses within the site are several orders of magnitude more permeable than the fines. The difference in the rate at which these materials will accept infiltrating fluids will greatly complicate the design of an infiltration network.

The perched aquifer beneath this site is relatively thin, ranging from four to eight feet in thickness, and exhibits seasonal fluctuations. Cross Section C-C', shown in Figure 2, illustrates the perched aquifer, as well as the correlatable Navarro Formation beneath it, and the uncorrelatable alluvial deposits above it. Although the aquifer is small, water levels beneath the site may have to be raised for effective treatment. Because the water-bearing materials are likely to hydraulically connect with the adjacent surface waters, some form of hydraulic or physical barrier, such as an extraction well system or a slurry wall, would be required to contain the groundwater for treatment.

Contaminant Profile

Priority pollutant analyses of soil and groundwater samples indicate extensive contamination with a wide variety of organics and inorganics. Table 1 summarizes results of analyses for organics conducted on soil samples from 5 boreholes and water samples from 3 wells. This is not a complete listing of organics detected, but lists some representative compounds present at high concentrations, and the sum of some compounds by class.

The principle priority pollutants found in soil samples at levels over 10 ppm include 1,2-dichlorobenzene and 1,4-dichlorobenzene. Priority pollutants found in groundwater include tetrachloroethylene (PCE) as high as 15 ppm, trichloroethylene (TCE) as high as 6.7 ppm, and trans-1,2-dichloroethylene as high as 7.7 ppm.

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*The hydrogen peroxide used was Restore™ 105 Microbial Nutrient supplied by FMC Aquifer Remediation Systems.

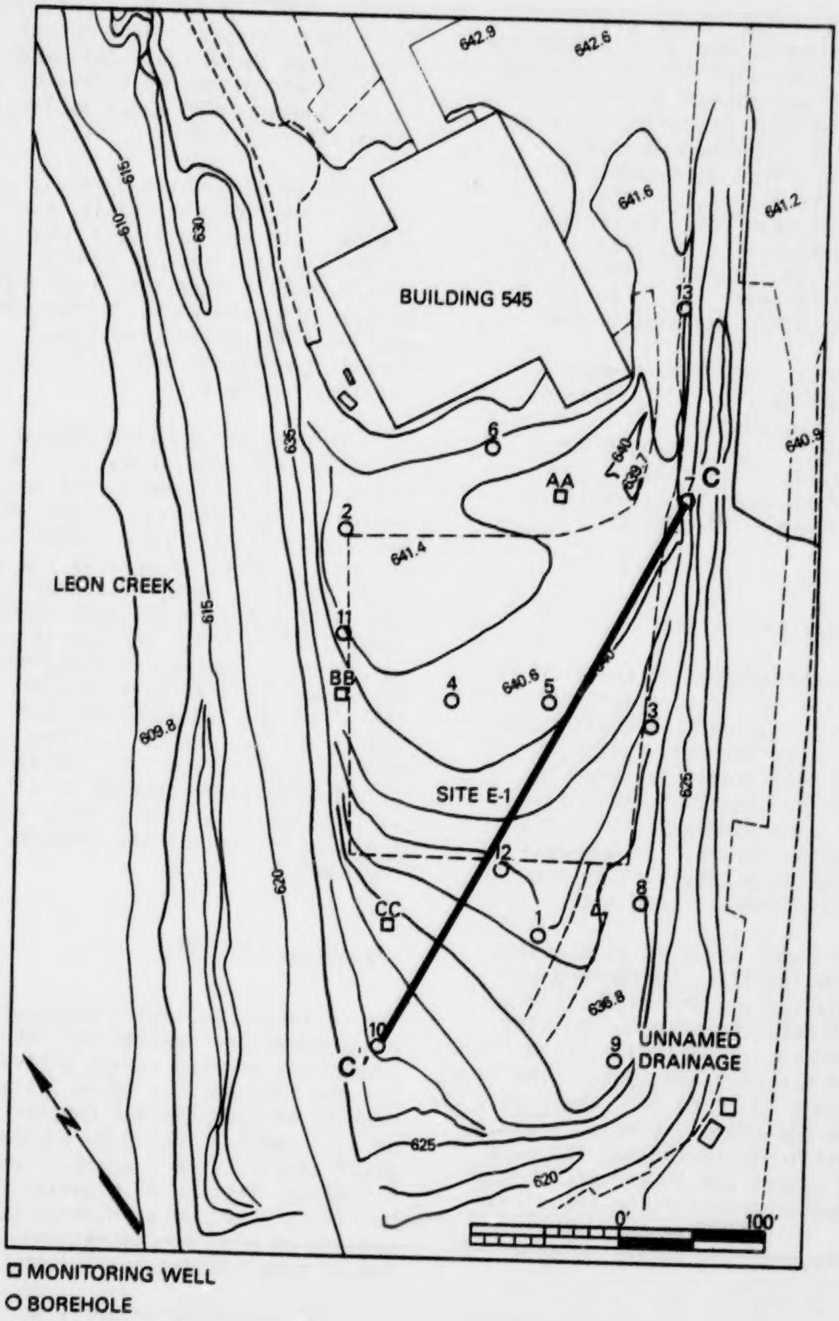
642.9
642.6
641.6
641.2
BUILDING 545
610
615
630
635
13
6
640
639.7
AA
C
7
640.9
2
641.4
LEON CREEK
615
11
BB
4
640.6
5
640
3
609.8
SITE E-1
625
620
12
8
CC
1
4
636.8
10
C'
9
UNNAMED DRAINAGE
N
625
620
0'
100'
□ MONITORING WELL
○ BOREHOLE

Figure 1. Site Plan with Cross Section Location.

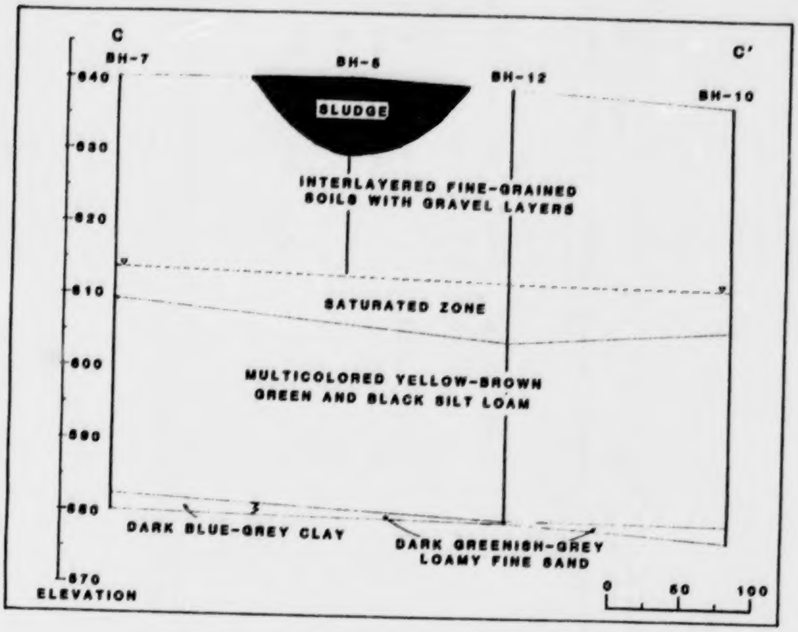
C
BH-7
BH-5
BH-12
C'
BH-10
640
630
620
610
600
590
580
570
ELEVATION
SLUDGE
INTERLAYERED FINE-GRAINED SOILS WITH GRAVEL LAYERS
SATURATED ZONE
MULTICOLORED YELLOW-BROWN GREEN AND BLACK SILT LOAM
DARK BLUE-GREY CLAY
DARK GREENISH-GREY LOAMY FINE SAND
0
50
100

Figure 2. Cross Section C-C'.

Petroleum-based hydrocarbon concentrations were particularly high in soil samples from the shallow sampling depth of borehole 5 and the deep sampling depth of borehole 1, at concentrations of 879 ppm and 221 ppm respectively. Groundwater from well AA contained 3595 ppm hydrocarbons.

The main inorganics present in all soil samples at concentrations greater than 10 ppm are antimony, chromium, copper, lead, nickel, and zinc. In 3 of the soil samples analyzed chromium concentrations exceeded 340 ppm. Cadmium and silver were present as high as 8 ppm, and mercury as high as 0.3 ppm. Although concentrations in groundwater were generally low and did not exceed 1 ppm, analyses indicated contamination levels slightly exceeding the National Interim Primary Drinking Water Standards for cadmium, chromium, and lead.

The distribution of contaminants throughout the site is extremely heterogenous and a defined plume of contamination has not been identified. However, based upon the high concentrations of organics, particularly hydrocarbons, present in the samples of boreholes 4 and 5 and at depth in borehole 1, it is surmised that migration is occuring, carrying contaminants from the former evaporation pit around boreholes 4 and 5 into the saturated zone in the area of borehole 1 (Figure 1).

Microbiological Investigations

Subsurface samples were taken and shipped to the Memphis State University laboratory for direct and viable cell counts. Acridine orange direct cell counts were performed according to the method of Ghiorse and Balkwill (1). Estimates of viable bacteria density were made using dilution-spread plate procedures on seven media: two nutient rich, two low-nutrient, and three dilute groundwater agar. Direct count estimates ranged from 7.6×10^6 to 1.7×10^8 cells per wet gram sample. Viable cell counts ranged from less than 100 to 7×10^6 colony forming units per wet gram sample. The data indicated that pollutants present did not reduce the number of total bacteria compared to other sites. Of particular note was the apparent non-selectivity of any of the seven media. All media yielded similar numbers of cells for any one

TABLE 1. REPRESENTATIVE CONTAMINANT CONCENTRATIONS, PPM

	Tetrachloro-ethylene	Trichloro-ethylene	1,1,-Trichloro-ethane	Trans 1,2-Dichloro-ethylene	Chloro-benzenes	Dichloro-benzenes	Alkyl-benzenes	Toluene	Ketones	Hydrocarbons	
										n-Alkanes	Branched
Soil											
Borehole #1	0.001	0.01	0.10	-	0.093	-	0.005	0.038	0.037	40	158.
" #2	-	0.016	0.97	-	-	0.15	-	0.005	0.30	-	-
" #3	0.022	-	-	-	-	-	-	-	1.0	-	0.009
" #4	-	0.91	-	-	-	0.16	0.020	0.022	2.2	58	0.11
" #5	0.019	0.082	-	-	0.012	38.	78.	0.13	26.	454	456.
Groundwater											
Well: AA	-	-	0.076	7.7	-	0.26	50	-	<1.0	1145	2450.
" BB	15.	0.12	0.12	0.13	-	0.005	-	-	<1.0	-	-
" CC	0.13	6.7	-	2.2	-	<0.001	-	-	<1.0	-	-

Note: Values represent highest concentrations found at each borehole. Three soil samples from each borehole were analyzed.

sample. This lack of selectivity may indicate the presence of highly adaptive populations of bacteria. There was little variation noted as a function of either site location, depth, or pollutant concentration.

Laboratory Biodegradation Study

The previously completed contamination profile and microbiological investigation were used in planning subsurface sampling for the laboratory biodegradation study. Subsurface samples were sent to the JRB laboratory, where both aerobic and anaerobic microcosms were assembled. The results of the lab studies as of the day 50 analysis indicate that biological degradation was occurring in the test microcosms. Significant differences were evident between the aerobic and anaerobic microcosms in terms of the nature of the compounds degraded. The patterns of degradation correlate well with the existing literature. Compounds which degraded in the aerobic microcosms include the hydrocarbons, aromatics, and the higher molecular weight halogenated organics such as chlorobenzene. Compounds which degraded anaerobically but not aerobically were most of the lower molecular weight halogenated hydrocarbons, including PCE and TCE. The compounds 1,1-dichloroethane and 1,1-dichloroethene disappeared by day 50 in both the aerobic and anaerobic microcosms, although rates were generally faster in the aerobic microcosms.

Table 2 presents selected results of the GC analysis of the base neutral/ acid extractable (BN/A) fractions of the microcosm soil samples. (Concentrations of organics in the microcosm water fractions were repeatedly so low that BN/A analysis of this fraction was discontinued). The results are characteristic of hydrocarbon biological degradation patterns. The concentrations of total resolved hydrocarbons, representing preferentially biodegraded n-alkanes, were reduced rapidly in the aerobic microcosms, and remained essentially unchanged in the anaerobic microcosms. Concentrations of total unresolved hydrocarbons, representing less preferentially degraded branched alkanes, were also degraded aerobically but remained unchanged in the anaerobic microcosms. The ratio of unresolved to

TABLE 2. CONCENTRATIONS OF HYDROCARBONS IN MICROCOSM SOIL FRACTIONS

	HYDROCARBONS, PPM					
	Total Resolved Hydrocarbons		Total Unresolved Hydrocarbons		Ratio of Unresolved to Resolved	
Day-Treatment	X*	CV**	X	CV	X	CV
1 - H_2O_2	20.3	14	205	22	10.1	10
24 - H_2O_2	14.0	19	176	16	12.6	5
49 - H_2O_2	6.91	23	102	22	14.8	3
1 - Oxygen	19.6	5	151	15	7.71	10
24 - Oxygen	9.78	23	170	16	17.7	15
49 - Oxygen***	3.52	67	71.0	61	20.9	8
1 - Anaerobic	12.5	33	89.5	27	7.26	9
25 - Anaerobic	14.4	57	132	51	9.60	12
50 - Anaerobic	13.5	25	119	35	8.69	13

* All values are the means of three replicates unless otherwise noted.
** CV: coefficient of variation. (Standard deviation/mean x 100).
*** Means of two replicates.

resolved hydrocarbons increased significantly in the aerobic microcosms, as the preferred n-alkanes were more rapidly degraded.

The preliminary available data indicate that in some cases the oxygen-treated microcosms appeared to exhibit more rapid rates of degradation than the hydrogen peroxide treated microcosms. The total resolved and unresolved hydrocarbons disappeared more rapidly in the oxygen-treated microcosms (Table 2). Oxygen-treated microcosms exhibited decreases in resolved and unresolved hydrocarbons of 82 percent and 53 percent, respectively. Peroxide-treated microcosms exhibited decreases in resolved and unresolved hydrocarbons of 66 percent and 50 percent, respectively. A similar trend was repeated for 1,1-dichloroethane, 1,1-dichloroethylene, and chlorobenzene (Figure 3). The trends illustrated for 1,1-dichloroethane and chlorobenzene suggest that an acclimation period of approximately one month may be required for the microbial population to adapt to the peroxide treatment. After an acclimation period, degradation appears to proceed as rapidly as in the oxygen-treated microcosms. Results from the day 100 analyses, which are not yet complete, are necessary to confirm these trends.

Anaerobic degradation resulted in the disappearance of most of the lower molecular weight halogenated hydrocarbons (Figure 4). 1,1-Dichloroethane, 1,1-dichloroethylene, and 1,1,1-trichloroethane disappeared by day 25 in the anaerobic microcosms. PCE and TCE had disappeared by day 50. The concentrations of trans-1,2-dichloroethylene increased, probably because it is being produced as a degradation product of PCE and TCE (2).

CONCLUSIONS

Biological degradation is a feasible treatment alternative and will be effective at the Kelly site if the metabolism of the indigenous microorganisms can be optimally enhanced. The limiting factors regarding site remediation include the nature of the biological degradation and the site hydrogeology.

The nature of the site stratigraphy and site hydrology will make it difficult to provide a uniform distribution of the permeating solution. The clayey soils have a low permeability. The distribution of contaminants throughout the site is fairly heterogenous and there does not appear to be any defined plume of contamination. The depth of the saturated zone is variable depending upon climatic

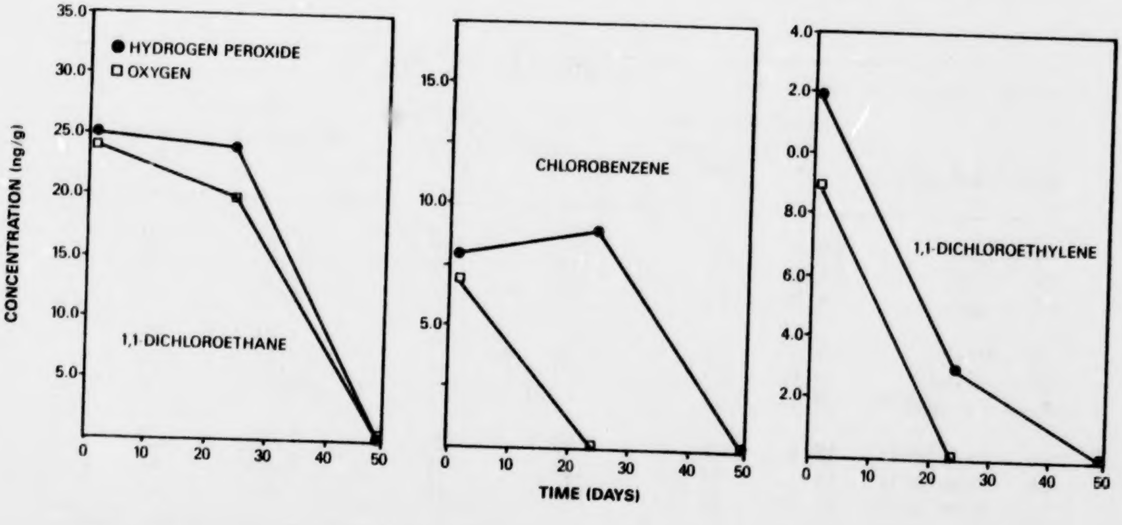

Figure 3. Change in Concentrations of 1,1-Dichloroethane, Chlorobenzene, and 1,1-Dichloroethylene with Time in Aerobic Microcosms. (Each data point represents a mean of three replicates.)

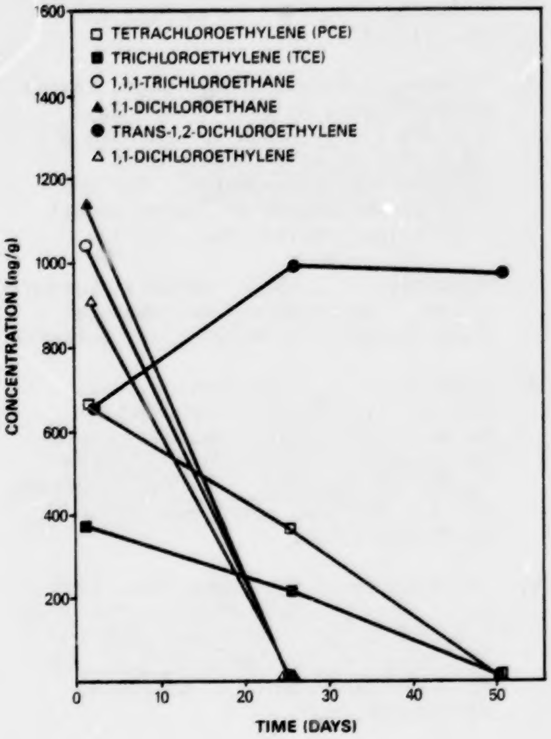

Figure 4. Change in Concentrations of Lower Molecular Weight Halogenated Hydrocarbons with Time in Anaerobic Microcosms. (Each data point represents a mean of three replicates.)

conditions, and there is a possible hydraulic connection of the groundwater with surrounding surface waters.

Aerobic biodegradation can be expected to reduce the hydrocarbon, aromatic, and halogenated aromatic concentrations. The hydrocarbons and unsubstituted aromatics will disappear most rapidly. PCE and TCE are not expected to degrade aerobically. For reducing the concentration of these lower molecular weight halogenated solvents, aerobic biological degradation will have to be coordinated with other treatments. It is unlikely that the metal concentrations at the site will be reduced without additional treatment. It is possible that the metals present will be mobilized, depending upon the treatment approach. This possibility will be investigated in further laboratory studies.

Metals are bound to the soil matrix in the form of organo-metal chelates and as sulfides. Under oxidizing conditions, particularly in the presence of a very strong oxidant such as peroxide, the reduced complexes may be oxidized. If the metals are not re-precipitated as ferric or manganic complexes, they may be mobilized into the groundwater. This would be particularly problematic if there is a leakage of groundwater into Leon Creek. (Leakage of peroxide/nutrient treated groundwater alone would not be problematic. The peroxide would rapidly decompose to oxygen and water, and the nutrients, at worst, could lead to an algal bloom.)

Aerobic degradation will be used at the site for two reasons. First, based on our laboratory studies anaerobic degradation was not as effective in degrading as wide a variety of contaminants, and second, aerobic degradation is much more easily accomplished in the field with a transfer of technologies from such incidents as gasoline spills.

Oxygen is apparently most easily and efficiently delivered to the subsurface in the form of hydrogen peroxide. Matters which must be resolved before hydrogen peroxide is used in the field are the possible mobilization of heavy metals, and possible precipitation of iron and manganese oxides and hydroxides and calcium and iron phosphates that could clog the injection wells.

Plans for the field demonstration will be formalized in early 1985. Currently we plan to design an injection well system near the center of the area to be treated, and an extraction well system surrounding this area. The treatment solution (hydrogen peroxide, nutrients and water) will be prepared in mixing tanks and added to the injection wells. Groundwater will be extracted at the site perimeter and recirculated to the mixing tanks. Monitoring will be conducted at the well head, at the mixing tanks and at other site locations to maintain an optimum treatment solution. Other monitoring points may include points on site to indicate progress and off site to indicate any problems with migration of pollutants or treatment solution. Our treatment goals at the site are to degrade (within the time constraints of our project) those contaminants shown to degrade in the laboratory. These contaminants include hydrocarbons, aromatics and halogenated aromatics. Following aerobic degradation, other treatment techniques, such as activated carbon filtration and/or air stripping may be needed to further address site problems.

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CASE STUDY INVESTIGATIONS OF REMEDIAL RESPONSE PROGRAMS AT UNCONTROLLED HAZARDOUS WASTE SITES

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ABSTRACT

Section 105(7) of the Comprehensive Environmental Response, Compensation and Liability Act of 1980 (CERCLA) requires that USEPA establish means of ensuring that remedial actions are cost-effective over the period of potential exposure to hazardous substances or contaminated materials. In partial support of this objective, USEPA has been directing the preparation of detailed case studies on remedial responses at uncontrolled hazardous waste sites. Detailed information has been collected on cost, institutional, and technical issues related to the implementation and effectiveness of remedial actions at these sites. Compiled into a succinct case study, this historical information can then be used in planning, selecting, designing, and cost analyzing future remedial actions.

The project discussed here is a continuation of earlier case study activities where 23 case studies were published (2). The focus of this more recent series of 8 case studies, however, was on fund-financed or enforcement actions, as opposed to the earlier studies which focused on private- and State-lead actions. Within this report, the 8 case studies are briefly described in table format, followed by a more comprehensive discussion of 4 of the 8 remedial case studies.

INTRODUCTION

As the Superfund program expands and the number of remedial responses increases, the nature and type of remedial action technologies implemented will continue to grow. It is also apparent that valuable lessons can be learned from the field implementation of remedial technologies at hazardous waste sites throughout the course of the Superfund program. As a result, it has become necessary, for programmatic and scientific reasons, to provide a systematic way of recording the results of remedial actions.

PURPOSE

A series of case studies of remedial actions completed at fund-financed and enforcement sites has been conducted. At selected sites, detailed case studies were performed to document the performance of applied remediation technologies, and to determine the limitations and applicability of these technologies in other remediation activities.

The results of the case study investigation are currently being used as an information transfer between those who have experience in using specific remedial actions, and those planning, assessing, or implementing site remediation efforts. The case studies have been prepared for an audience consisting of members of industry and commerce, state agencies, local authorities, and the USEPA.

Case study reports have been structured so that they provide detailed data on the remediation techniques employed; the circumstances and conditions under which they were implemented; their apparent effectiveness in correcting or controlling the problem; and their potential uses in other remedial action scenarios. The following is an outline of the typical case study report:

I. Introduction
  - Background
  - Synopsis of Site Response

II. Site Description
  - Surface Characteristics
  - Hydrogeology

III. Waste Disposal History

IV. Description of Contamination

V. Planning the Site Response
  - Initiation of Site Response
  - Selection of Response Technology
  - Extent of Site Response

VI. Design and Execution of Site Response

VII. Cost and Funding
  - Source of Funding
  - Selection of Contractors
  - Project Cost

VIII. Performance Evaluation

IX. Bibliography

APPROACH

Initially a survey was conducted of fund-financed sites and enforcement sites to identify candidate case study sites. The survey identified over 30 sites where remedial responses had been partially or entirely completed. The project team selected 8 sites for case study reports. The criteria used for selection included:

- Current status with regard to percent completion of the remedial action
- Availability, accessibility, and completeness of remedial action, cost and engineering data
- Type of remedial action technology implemented, so that a range of remedial action techniques were investigated
- Type of waste management practice so that a wide range of techn[illegible]gies, commonly used in hazardo[illegible] waste management, were studied
- Types of waste and contaminants present at the facility to ensure that a variety of waste streams and pollutants were included
- Hydrogeologic setting, so that a variety of settings were represented
- Enforcement sensitivity.

Once the case study sites had been identified, an in-depth and site-specific data acquisition effort was begun. All available background information relative to each site's environmental setting, operational history, remedial actions and Superfund involvement was collected. Two readily available sources of information were used to complete this task. The first source was site-specific information collected from public information data sources, including the following:

- USGS topographic and geologic maps, geologic and hydrologic reports, soils and climatological maps and reports
- USEPA and state files, including site-specific data on operational history, Superfund involvement, detailed remedial action, engineering design and cost data, previously documented cost studies, and litigative case support data.

The second source of site-specific data was direct correspondence with the USEPA Superfund on-scene coordinators, the facility operators, and the parties primarily responsible for designing and implementing the site remedial action. During correspondence, additional site-specific information was collected, and written or verbal approval for site visits was obtained. Prior to any field visits, collected information was thoroughly reviewed and data gaps identified for clarification during the site visits.

A schedule for all site visits was then developed. Follow-up telephone calls were made to appropriate contacts to coordinate field visit activities and schedules. Each site visit was two days in duration, and included the following activities:

- Detailed site inspection, including documentation of site layout, evidence of environmental contamination, and observation of remedial action technologies being employed
- Review of all available remedial action design drawings and existing cost information
- Detailed review of the remedial action data previously collected, including pre-remediation and monitoring activities, remedial action and treatment technologies, implementation and cost details, and remedial action design data.

All sources of information and site-specific data were carefully referenced in field logs for later documentation in the case study reports prepared for each site. Photographic slides or prints developed from the site visits were collected and developed to track the response history.

After every site visit, individual case study reports were prepared and forwarded to the USEPA Project Officer, Task Managers, and parties involved in implementing the remedial action for their review.

RESULTS

The 8 final case study reports describe an array of remedial technologies for implementation at uncontrolled hazardous waste sites. In addition to technical discussions, the case studies present available cost and funding data for each site, and describe the more important institutional issues that affected the implementation of the response programs. Table 1 summarizes the site conditions at the 8 sites prior to implementation of remedial response actions, the remedial technologies utilized, and the total estimated costs associated with the 8 response programs. The remainder of this section presents a more comprehensive discussion of the case study investigations conducted for the 4 sites at which remedial actions have been completed.

Gulf Coast Lead - Tampa, Florida

Gulf Coast Lead (GCL) is a small, secondary lead smelting facility whose operation consists of recovering lead from discharged lead acid batteries and lead groupings from battery manufacturers. Until 1979, rinsewater contaminated with sulfuric acid and heavy metals, and battery casings from plant operations, were deposited in an unlined surface depression near the center of company property, causing soil and shallow groundwater contamination. Surface runoff from the battery casing area threatened to spread the contamination to off-site areas, while the underlying Floridan Aquifer, Tampa's principal source of drinking water, was also threatened. A federal suit filed in October 1980 under RCRA resulted in a consent decree signed later that same month. Under this agreement between EPA and GCL, the remedial response at the site was completed.

The pollution problem at the Gulf Coast Lead facility was discovered in December 1978 by the Hillsborough County Environmental Protection Commission during an effort to bring all secondary lead smelting and battery recycling operations in the county into compliance with State and local regulations. On November 1, 1979, after receiving citizen complaints regarding GCL's noncompliance with State water quality regulations, USEPA's Surveillance and Analysis Division conducted a groundwater investigation. As a result of this investigation, groundwater from two on-site monitoring wells was found to contain high inorganic contaminant concentrations, metals above the Maximum Concentration Levels established under the Safe Drinking Water Act (SDWA), and lowered pH. Dead and discolored vegetation was found onsite; and significant amounts of lead, nickel, antimony and sulfuric acid were found in

TABLE 1. SUMMARY OF CASE STUDY INVESTIGATIONS

SITE	SITE CONDITIONS	SITE RESPONSE	STATUS OF REMEDIAL PROGRAM	ESTIMATED COSTS
Bruin Lagoon, Bruin Borough, Pennsylvania	Earthen-diked lagoon constructed in 1930's and used as a repository for oil production wastes, waste motor oil re-refining residues, coal fines and fly ash; began to draw public attention following a 3,000-gallon sludge spill into Bear Creek which lies adjacent to site.	● Lagoon embankment improvements ● Construction of concrete retaining wall along Bear Creek ● Tank demolition, removal and offsite disposal ● Removal and off-site disposal of liquid supernatant contained in lagoon ● Sludge stabilization ● Installation of venting system for acidic gas	I	$1.76 million
Denney Farm, Aurora, Missouri	Estimated 80-90 drums of wastes containing dioxins discarded in a trench on property known as Denney Farm.	● Excavation of waste materials and contaminated soils ● Placement of soils with visible contamination in drums ● Placement of other contaminated soils in microbiological degradation basins ● On-site storage of drummed wastes (solid waste and contaminated soils) ● Trench backfilling and capping ● On-site incineration for ultimate destruction and disposal of wastes currently under investigation	I	N/A

I - Incomplete
C - Complete
N/A - Not Available

(continued)

TABLE 1. (continued)

SITE	SITE CONDITIONS	SITE RESPONSE	STATUS OF REMEDIAL PROGRAM	ESTIMATED COSTS
Gulf Coast Lead, Tampa, Florida	Contaminated rinsewater and battery casings from lead recovery operations at the GCL smelting facility were dumped in unlined surface depression on company property. Soil and shallow groundwater became contaminated with sulfuric acid and heavy metals, thus presenting a threat to an underlying aquifer.	• Implementation of groundwater monitoring program • Removed battery casings from surface depression • Capping of site • Installation of retention basin to collect and recycle contaminated surface runoff from site • Installation of underdrain system to collect rainwater runoff from uncontaminated areas for discharge to Tampa storm drains • Installed a groundwater treatment system • Subsurface acid reaction barrier to intercept and neutralize contaminated groundwater that had migrated off-site was under study at the time of this writing	C	$700,000
Lipari Landfill, Pitman, New Jersey	Seven-acre area used for disposal of industrial and domestic wastes including solvents, formaldehyde, paints, phenol and anine wastes; concern focused on contamination of two nearby streams, a lake and two aquifers.	• Construction of a soil-bentonite slurry trench cutoff wall around a 16-acre area which included the 7-acre landfill • Installation of synthetic membrane cap over the 16-acre site • Installation of passive gas collection and venting system • Completion of groundwater treatability study and design of a collection and treatment system	I	$2.5 million

I - Incomplete
C - Complete
N/A - Not Available

TABLE 1. (continued)

SITE	SITE CONDITIONS	SITE RESPONSE	STATUS OF REMEDIAL PROGRAM	ESTIMATED COST
Picillo Farm, Coventry, Rhode Island	A 7.5-acre area used as a hazardous waste disposal site during the 1970's; over 10,000 drums containing industrial solvents and oils, pesti cides, PCBs, paint wastes and explosives were buried in 4 trenches on site; leachate migrated through soil into nearby swamp, contaminating groundwater, surface water and soil in vicinity.	● Drum removal and off-site disposal ● Ground and surface water monitoring of swamp and contaminated groundwater plume ● Detonation of unidentified, explosive materials ● Landfarming of contaminated soils ● Completion of a feasibility study on groundwater recovery, treatment and discharge	I	$4.96 million
Rose Park Sludge Pit, Salt Lake City, Utah	Six-acre disposal site located in a public park; between 1920 and 1957 acid sludges were discarded in unlined pits and covered with lime and soil; the site was covered with a soil cap in 1960; in 1976 construction activities at the site forced sludge to the surface; low concentions of contaminants were found in groundwater immediate area but had not migrated appreciably.	● Installation of groundwater monitoring wells around sludge pit ● Construction of a soil-bentonite slurry trench cutoff wall around the site ● Construction of a sand layer, filter fabric and clay cap system ● Installation of fence around perimeter of site	C	$1.2 million

I - Incomplete
C - Complete
N/A - Not available

(continued)

TABLE 1. (continued)

SITE	SITE CONDITIONS	SITE RESPONSE	STATUS OF REMEDIAL PROGRAM	ESTIMATED COST
Tacoma Well 12A, Tacoma, Washington	Well 12A, the northern most of 13 wells in a well field in South Tacoma Channel, was taken off line after chlorinated organic solvents were detected in a groundwater sample; the source was identified as being north of the well field; the spreading of contaminants to other wells after 12A was taken out of service indicated that further contamination of the well field could be prevented if Well 12A remained operational to act as an intercepter or barrier.	• Installation of an air-stripping system at Well 12A to remove volatile organics (considered an interim remedial measure) • Completion of studies to identify contaminant source, and design and implement a permanent solution	C	$948,133
Taylor Road Landfill, Seffner, Florida	A 42.5-acre sanitary landfill in which unknown quantities of hazardous wastes including solvents, paint thinners, insecticides, herbicides, fungicides and sludges were deposited; groundwater samples collected at the site were found to contain VOCs and metals in concentrations above acceptable safe drinking water standards.	• Upgrading of surface cap • Channelization of surface drainage along eastern and southern site boundaries • Installation of a methane gas collection and control system • Development of long-term groundwater and methane gas monitoring program	C	$3.56 million

I - Incomplete
C - Complete
N/A - Not Available

the shallow groundwater. A later investigation confirmed the source of the problem as acid infiltration from untreated or partially treated wastewater discharged into the shallow aquifer, and metals being leached from the surface into the groundwater.

In order to immediately reduce the threat to the local private water supply caused by the discharge of untreated or partially treated wastewaters, GCL was ordered to upgrade its treatment system to meet the influent requirements of the Tampa Publicly Owned Treatment Works (POTW). Hence, discharge to the unlined surface depression was halted. The consent decree signed between EPA and GCL in October 1980, then required that GCL either eliminate, or isolate and contain, all hazardous constitutents in the groundwater attributable to GCL operations.

The selected remedial plan at the GCL site consisted of capping, combined with groundwater withdrawal and treatment. This plan provided the greatest assurance that further contamination in the area would be eliminated. This was at a later date augmented, through negotiations with the EPA, by the addition of an "acid reaction barrier" to intercept and neutralize contaminated groundwater that had already migrated offsite.

The remedial plan implemented at GCL consisted of the following remedial technologies:

- An earthen cap over part of the unpaved area of the site and an asphalt cap over the remainder
- A surface water control system
- A groundwater withdrawal and treatment system
- A long-term groundwater monitoring program
- A subsurface acid reaction barrier.

The installation of these technologies is briefly discussed below.

Capping -

The objective of this remedial technology was to prevent further infiltration of rainwater. Therefore, all unpaved areas at the site were capped with either asphalt or clay. The clay cap was placed over a 3.4-acre area of the site overlying the buried battery casings. This cap consisted of a 4-inch layer of compacted clay over the existing grade, followed by 8 feet of topsoil and grass sod.

Surface Water Controls --

Surface water controls were implemented to collect the increased runoff from the cap surface. The surface water control system consisted of two main components: a stormwater detention pond and a stormwater detention basin. The stormwater detention pond is lined with .035-inches of reinforced chlorinated polyethylene. It receives all rainwater falling on the paved areas of the GCL plant. In addition, the pond receives runoff from sprinkling systems located on all buildings which are used to reduce the levels of lead in the air. The water collected in the pond (called graywater) is recycled for use in the sprinklers and in the battery washing system. The stormwater detention basin is designed to collect surface runoff from the clay cap for conveyance to the City stormwater ditch which parallels the northern boundary of the GCL site.

Groundwater Withdrawal and Treatment --

The objectives of the groundwater withdrawal and treatment system are: (1) to contain (by reversal of the groundwater gradient) the off-site migration of contaminants, and (2) to eliminate the sources of the contamination. Pumping was, therefore, implemented in two areas.

One area was directly above the battery casing disposal location and the other area was along the western and southwestern boundaries of the GCL site. The battery casing area was selected because it was the source of groundwater contamination, and pumping would result in the quickest improvements in groundwater quality. The western and southwestern boundaries of the site were selected for groundwater withdrawal because pumping these areas would intercept contaminated groundwater flowing away from the site, and also would draw contaminated groundwater offsite and into the treatment system.

Withdrawn groundwater is treated in GCL's wastewater treatment system, which consists of primary clarification, pH adjustment, and final clarification, and then discharged to the Tampa POTW.

Groundwater Monitoring --

The groundwater monitoring program at the GCL site was designed to detect any unsuspected migration of contaminants within the upper aquifer, and to detect any downward movement of contaminants into the Floridan Aquifer. These objectives are being met through quarterly sampling of 6 upgradient and 21 downgradient wells at the site.

Subsurface Acid Reaction Barrier --

The purpose of the subsurface barrier is to intercept and neutralize any contaminated groundwater that had already migrated offsite. At the time of this writing, studies were being conducted to determine the proper depth and placement of this barrier which will consist of a deep trench filled with oyster shells and crushed limestone.

Cleanup Objectives and Effectiveness --

The main objective of the response effort was to successfully isolate and contain hazardous contaminants attributable to company operations that were found in the soil and shallow groundwater beneath the site. In this way, the threat of contaminating the Floridan Aquifer, and thus, area drinking water supplies, would be eliminated. Capping the site, which was one element of the stormwater management strategy, served to minimize infiltration of rainwater and thus prevent displacement of the contaminated fluids. It also served to isolate the source of contamination and partially mitigate threats to public health and the environment. Pumping and treating the contaminated groundwater directly also served this purpose. In total, the remedial actions are expected to correct the contamination problem at the site. The remedial plan will be considered successful when all wells onsite achieve acceptable water quality levels based on SDWA Maximum Concentration Levels.

The effectiveness of the remedial response technologies at the GCL site cannot be evaluated at this writing. However, the selection of these response technologies appears to have considered all viable alternatives, and the operation of the response equipment has proceeded as expected. Unless major problems arise, such as clogging of the extraction wells, improvements in the extracted groundwater should be apparent within a 5-year period.

Rose Park Sludge Pit - Salt Lake City, Utah

The Rose Park Sludge Pit is a former waste disposal site located on 5 to 6 acres of a 33-acre public park in an industrial/residential area of northwest Salt Lake City, Utah. From the 1920's until the mid 1950's, an undocumented amount of acid waste sludge from petroleum refinery operations at the Utah Oil and Refining Company was discarded into unlined pits onsite and then covered with lime and soil. The waste acid sludges were generated from the acid treatment of light petroleum oil products at the refining company. Remedial work at Rose Park was funded as a private party response.

In 1957 Salt Lake City purchased the site in response to residents' complaints about the dumping. An estimated 40 to 100 truckloads of sludge were excavated and transported to a local dump, after which the area was covered with soil and left undisturbed for almost 20 years.

During a 1976 expansion of park facilities in the area, plans to build a Little League ball field and tennis courts were halted when a bulldozer broke through the overburden cap and started the effusion of sludge from the site. Workers began to complain of respiratory and eye irritations and headaches due to sludge fumes, prompting the Salt Lake City/County Health Department to analyze samples of the sludge. In late 1977, the Utah Geological and Mineral Survey (UGMS) began a preliminary investigation of groundwater contamination at the site, and in July 1979 USEPA Region VIII visited the site.

Analysis of the sludge by the County Health Department revealed that the sludge was acidic and that the mist released was sulfuric acid. The UGMS study concluded that the site was possibly contributing oil and grease contamination to the groundwater downgradient from the site. During the EPA preliminary assessment, chemical analyses of the sludge samples indicated the presence of various pollutants including xylene, o-dichlorobenzene, pyrene, fluoranthrene and phenanthrene.

The primary immediate hazard at the Rose Park site was the risk of exposure through direct contact with the waste materials by humans. Due to the viscous nature of the waste material, contaminant migration potential was very low, and the direct exposure route was a primary concern in developing the remedial action. Short-term measures taken to reduce the immediate hazard of direct contact were the installation of a 6-foot high chain-link fence with a barbed wire top rail, extension of the property boundary by approximately fifty feet and new signs warning of the hazard. A detailed montoring program to improve characterization of the problem was then implemented.

The remedial response actions agreed upon in an Intergovernmental/Corporate Cooperative Agreement between the State and the responsible private party, provided for monitoring, capping, slurry wall construction, and security measures. The implementation of these actions are briefly described below.

Groundwater Monitoring --

Sampling by EPA of all 10 monitoring wells installed at the site will occur quarterly in the first year of remedial action operation. After year one and through year 30, sampling of only six of the wells becomes the responsibility of the City and the private party. The sampling program is intended to ensure that the remedial action is operating properly over its effective life.

Slurry Wall --

Final specifications called for construction of a soil-bentonite slurry wall which extended to a depth of about 30 feet into the underlying low permeability silty clay, and around the 2,326-foot perimeter of the sludge pit. Specifications called for the use of Wyoming-type bentonite which conformed to American Petroleum Institute specification 13A - Oil Well Drilling Fluid Material (3).

The main trench was excavated to a depth of about 30 feet. This allowed the wall to penetrate at least 10 to 15 feet into the underlying low permeability clay soil, 15 feet below the lowest observed oily residue zone and 10 feet below the deepest known extent of the sludge pit. The completed trench was a minimum of 3 feet and a maximum of 5 feet thick.

Capping --

The second phase of the remedial action involved placement of a cover over the sludge pit. The components of the cover construction included the following: cleaning and site grading; placement of bank run fill, clean sand fill, filter fabric, clay cover, and topsoil; and revegetation of the disposal area.

Site grading was necessary to achieve a 2 percent minimum grade. Following the site grading activity, a sand layer and gas vents, designed to provide a pathway through which gases could be atmospherically vented, were installed. Filter fabric was then placed loosely over the sand fill to provide stability and to prevent upward migration of contaminants. A 2-foot thick clay cap of a low permeability silt was then placed over the site, followed by 8 inches of topsoil. Revegetation was postponed until weather conditions were favorable during the growing season, and has since been completed.

Site Security --

Prior to vacating the site in July 1984, the chain link fence to the north of the site, which had been removed at the start of the trenching operation, was reinstalled.

A vehicular barrier was erected in October 1984. The barrier is a corrugated steel rail mounted on steel posts and stands about 3 feet high. It runs along the west, south and east sides butting into the chain link fences on the north. Warning signs have been placed at each entrance prohibiting digging or vehicular traffic.

Cleanup Objectives and Effectiveness --

The primary objective of the remedial action at the Rose Park site is containment of the waste material. The purpose of containment is to remove the threat of direct physical contact with the sludge and its health-threatening components. The sludge reportedly has low potential for migration and does not pose an offsite hazard to health and the environment. However, the selected remedial action considered the flow of groundwater through the site and migration of the contaminants, in addition to removing the threat of direct physical contact. Monitoring of the site to insure the integrity of the containment system was accomplished with wells provided by a private party.

Available information indicates that the Rose Park remedial actions were carried out according to plan and have been successful. The containment system greatly reduces the potential for direct contact, as well as providing the added benefit of eliminating odors. Further, the containment system has greatly reduced the possibility of contaminant migration, and further decreased the possibility of groundwater contamination.

Tacoma Well 12A - Tacoma, Washington

The Tacoma Well 12A site is located in Tacoma, Washington, a historically industrial area. During the summer of 1981, chlorinated organic solvents were detected in the water from City Well 12A, one of 13 in a well field used to meet increased demand for potable water during summer months. The source of contamination at the well field has not yet been conclusively identified, but a 1983 study revealed that it may be waste sludges from an oil reclamation company that was located in the area but has since gone bankrupt and is no longer in operation. Enforcement actions began with sufficient information to link a specific party to the pollution, USEPA and the State entered into a Superfund Cooperative Agreement for remedial work in June 1982.

The sequence of events leading to the discovery of the contamination problem at Well 12A began with routine sampling and testing of Tacoma well water during the National Organic Monitoring Survey in March 1976. Traces of trichloroethylene were found, prompting the Washington Department of Ecology (DOE) and the Tacoma/Pierce County Health Department to initiate an investigation of possible source(s) of the contamination. When over 600 drums of stored chemicals were found at the American Surplus Company (buyers and sellers of chemical solvents and paint thinner), only two blocks from Well 12A, EPA and the Washington DOE decided to sample Well 12A.

Results of this sampling indicated the presence of such chlorinated organic solvents as 1,1,2,2-tetrachloroethane, trichloroethylene, 1,2-transdichloroethylene and tetrachloroethylene. The Washington Department of Health and Human Services concluded that the use of Well 12A for drinking purposes would constitute a definite health threat, and subsequently removed the well from service. However, the same pollutants that were found in Well 12A began to show up in other City wells and monitoring wells constructed during the remedial investigation.

The need for potable water in the area could not be met by simple conservation methods or other alternatives, so abandoning the well field was not considered a viable option. Consequently, an Initial Remedial Measure was developed to contain the contaminated plume and to provide safe drinking water, until a more permanent solution could be found following completion of additional source identification activities. Pumping and treatment of the water from Well 12A was the selected response to the health hazard posed by the contaminants. Pumping Well 12A at an appropriate rate prevents contaminant migration to other wells in the field by altering the hydraulic gradient. In addition, water withdrawn from Well 12A is treated and used for water supply purposes during summer months.

Of the treatment alternatives considered, air stripping and carbon adsorption treatment were considered viable responses. Data from bench scale and pilot plant studies of these technologies indicated the air stripping was more economical and efficient at the scale required for Well 12A. The air stripping system installed at the Well 12A site is briefly discussed below.

Air Stripping Treatment System --

The pumping/air stripping system was designed as a temporary measure to provide necessary drinking water and control further pollutant migration. A design life of five pumping seasons was selected, although actual operation of the system will be dependent on the outcome of further studies and the implementation of a permanent cleanup plan.

A pumping rate for Well 12A of 3500 gallons per minute (gpm) results in the desired hydraulic gradient, which acts as a barrier to further migration of contaminated groundwater to other wells in the field (which, without pumping of Well 12A, would be downgradient of Well 12A). Five air stripping towers, with design capacity of 700 gpm each, are used in parallel to treat the total flow of 3500 gpm. The air flow rate necessary for stripping is 29,000 standard cubic feet per minute per tower. An air permit was obtained from the Puget Sound Air Pollution Control Agency for the emission of volatile organic compounds stripped from the withdrawn groundwater.

Cleanup Objectives and Effectiveness --

The objectives of the treatment system enacted at Tacoma Well 12A were:

- To block the spread of contaminants
- To supply drinking water within applicable water quality standards
- To partially clean up the aquifer.

To block the spread of contaminants, the treatment system for Well 12A had to operate over extended periods of time either discharging to Commencement Bay or to the Hood Street Reservoir when the well field was needed as water supply in the summer months. In order to operate over a period of time and to meet applicable water quality standards, discharge limits were placed on treated effluent as follows:

- A maximum of 15.5 pounds/year of 1,1,2,2-tetrachloroethane could be discharged to the Hood Street Reservoir.
- The average concentration of effluent to Commencement Bay of 1,1,2,2tetrachloroethane had to be 50 ppb or less.
- The maximum concentration allowed for discharge to Commencement Bay was 100 ppb.

Therefore, the treatment system had to operate so that less than the maximum quantity of 1,1,2,2-tetrachloroethane would enter Hood Street Reservoir during the period when the well field was operating, and so that effluent limits would be within acceptable ranges when treated effluent was discharged to the bay. If the 15.5-lb limit was reached, water could no longer be discharg ed to the drinking water system, and if the concentration limits were exceeded, the well would be forced to shut down.

The efficiency of the Well 12A treatment system was approximately 98 percent during the period from August 4, 1983 to November 4, 1983. Calculations made in October 1983 showed that effluent from the well could be sent to the Hood Street Reservoir for 305 days before the 15.5-lb limit was reached. In addition, the effluent discharged never rose above the 50-ppb limit set as the average for discharge to Commencement Bay. This record of performance indicated that the system could be operated for periods much longer than required to protect other wells and still be within the discharge limits. Because the effluent concentrations were low, treated water could be discharged to the Hood Street Reservoir and into the City's distribution system for a period much greater than required to meet drinking water supply needs. Therefore, the system accomplished two of the three stated objectives:

- The system permitted the well to operate for periods while the well field was in operation, thus blocking the spread of contaminants.
- The system treated Well 12A groundwater such that effluent could be discharged to the Hood Street Reservoir if needed.

The system appears to be meeting its third objective--partially cleaning up the aquifer--but sampling data, at the time of this writing, was only available from a single pumping season.

Taylor Road Landfill - Seffner, Florida

Taylor Road is a 42.5-acre landfill, which had been owned and operated by Hillsborough County near Seffner, Florida, approximately 10 miles east of Tampa. Although originally intended only for the disposal of municipal refuse, the landfill had unknown quantities of solid and hazardous wastes including solvents, paint thinner, halogens, insecticides, herbicides, oil, sludges, fungicides and other wastes deposited there between 1976 and 1979. Improper site management and disposal operations contributed to the areal groundwater, soil and air contamination which was attributed to Taylor Road and one of two other Countyowned waste repositories adjacent to the site. A suit was filed under RCRA and SDWA in October 1980 and later amended to include CERCLA. The case was settled in June 1983 when Hillsborough County, the USEPA and the Florida Department of Environmental Regulation (DER) signed a consent decree for a $2 million remedial program.

In early 1979, the Taylor Road Landfill site came to the attention of USEPA Region IV personnel when area residents expressed their concern about a proposed expansion of the existing landfill by the County. The site is situated near several private residences and within a 1.5-mile radius of various commercial, industrial and institutional establishments. In early August 1979, EPA scheduled a site inspection in response to the numerous citizen complaints and as part of the nationwide, Congressionally mandated inventory of landfills.

Analysis completed in December 1979 by EPA's Water Supply Branch revealed that soil and monitoring wells were heavily contaminated with organic compounds and heavy metals, and that potable wells also contained traces of various volatile organic compounds (VOCs). Similar sampling by County consultants supported EPA's findings. A March 1980 preliminary assessment by EPA, documents continued contamination of the water supply, noticeable odors, and evidence of unknown amounts of solvents and hospital wastes buried at the site. Tests conducted on August 20, 1980 by a team comprised of Hillsborough County and DER personnel, found methane gas offsite at concentrations exceeding the lower explosive limit even though a preliminary gas venting system had previously been installed.

Following the determination by EPA in 1979 that groundwater in the vicinity of Taylor Road was contaminated, EPA sent warnings to residents and businesses in the area that were using private wells for their water supply, to discontinue use of all wells, and contacted State and County officials to recommend that they provide alternate supplies of water to these residents and businesses. Hillsborough County immediately initiated delivery of bottled water to affected well users to abate the hazard posed by use of the contaminated groundwater, and has plans for extending County water service to these users.

A remedial investigation was begun in November 1981. This study was intended to evaluate the site, and confirm the presence and extent of contamination, evaluate existing pollution controls (i.e., cap, drainage ditches, and gas control system) and recommend remedial alternatives for the mitigation of groundwater contamination originating from the landfill.

The selected remedial alternative was the installation of the impermeable cap and surface drainage system. The system was selected because it was technically feasible, did not jeopardize the existing integrity of the Taylor Road or adjacent landfills, and had essentially no human health or environmental risk associated with its implementation.

The response action at the Taylor Road Landfill site was comprised of two phases: (1) upgrading of the facility cap, and (2) improving surface drainage to prevent cap erosion and water infiltration. Monitoring at the site was also required to detect any future problems and to determine the effectiveness of the response actions. The implementation of the selected response actions are briefly described below.

Cap Upgrading --

The consent decree mandated that about 65 percent of the surface area of the site be upgraded. Permeability tests were to be performed on an additional 6 acres, and if the permeability was not acceptable, then these areas would also have to be upgraded.

Areas requiring upgrading were to be improved such that the entire area of the site would be covered by a minimum of 2 feet of clay soil with a permeability of 1 x 10^{-6} cm/sec or less. Upgrading was accomplished by removing topsoil and subsoil on the existing cover, mixing subsoil with a montmorillonite clay, and replacing and compacting the soil-clay admix to meet permeability specifications.

Due to capping with impermeable clay (which prevents venting of gases through the landfill surface), a gas collection and venting system was required to prevent horizontal migration of methane generated by degradation of organic wastes. The gas vent system is composed of 42 collection wells, header pipes, condensation traps, and a blower and flare station. The header system connects all wells to the blower and flare station. Sample points, condensation traps and control valves are located throughout the system to allow for maintenance control and inspection of the system. The County has been initially flaring the methane gas but eventually intends to recover and use the methane.

Surface Runoff Control --

Surface runoff control from the cap was improved to prevent ponding and erosion. Clay-lined drainage channels were constructed or upgraded around the cap perimeter. The original clay linings were found to erode relatively quickly due to the heavy rainstorms that typically occur in the area, and the clay linings were replaced with concrete by the County to minimize erosion and future maintenance costs. Additionally, the County installed concrete flumes along the side slopes to channel water from the surface down the sides of the fill with minimum erosion.

Monitoring Systems --

To insure that the initial remedial actions were effective, a detailed monitoring program for groundwater and methane gas was implemented. Groundwater samples are collected quarterly from 4 wells, and semiannually from 3 additional wells. Additionally, three new wells may be installed for additional monitoring. Flexibility in monitoring allows for the relocation of monitoring points but not for increase in the number of points. In turn, this allows for the accurate monitoring of the groundwater if the flow direction changes.

A methane gas monitoring system was established around the Taylor Road Landfill and two adjacent County-owned waste repositories, for the purpose of detecting the migration of methane gas beyond the boundaries of the fill area. This system consists of 32 wells extending from surface to bedrock.

Cleanup Objectives and Effectiveness --

The overall short term objective of the remedial action program at Taylor Road was to control the migration of existing contamination as well as to prevent new contamination. The response was designed to minimize surface water infiltration and the migration of methane gas. The long term remedial action goal is to ensure that the system is effective and that future contamination is prevented, thereby eliminating the risk of exposure to area residents from a polluted drinking water supply.

There are no data currently available to evaluate the effectiveness of the remedial actions. However, based on the site conditions and information that is available, the remedial actions selected and implemented should be effective in the long-term. If properly maintained, the systems should effectively control leachate from the Taylor Road Landfill by preventing infiltration, which prevents the displacement of leachate currently in the landfill and prevents formation of additional leachate.

Summary

Together, the 8 case study reports describe a variety of remedial technologies that have been implemented at fund-financed and enforcement action sites. The initial environmental conditions, i.e., hydrogeology, type of contamination, contaminant levels, population affected, etc, that characterized each site were unique, as were the circumstances and events leading to each site's discovery and remediation (see Table 1). The usefulness of the information contained in these reports lies with this characteristic that each report presents an entirely unique set of circumstances under which the regulatory authorities (Federal, state, and local), and the public sector had to act in order to design a resolution to a problem that posed a threat to the environment and public health. The case study reports provide detailed accounts of remedial program design, initiation and implementation. They also include brief evaluations of the effectiveness of completed remedial response programs. With the type of information contained in these reports, they can serve as useful learning tools for those involved in all phases of remediation programs implemented at uncontrolled hazardous waste sites.

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EFFECT OF FREEZING ON THE LEVEL OF CONTAMINANTS IN UNCONTROLLED HAZARDOUS WASTE SITES - PART I: LITERATURE REVIEW AND CONCEPTS

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ABSTRACT

A literature search indicated that natural freezing may have detrimental effects at uncontrolled hazardous waste sites in the cold-dominated areas because of frost action on buried materials and ion movement in soils. Natural and artificial freezing, however, can be used beneficially to concentrate effluents, and to dewater sludges, contaminated sediment and soils. The process of artificial ground freezing can also be used as an alternative to temporarily immobilize contaminant transport and potentially for decontamination of soils, sediments and sludges. A cost and economic analysis procedure was developed and used to evaluate ground freezing.

INTRODUCTION

Hazardous waste treatment and disposal is one of the major environmental concerns currently facing the nation. The amount of waste generated annually in the United States is estimated to be 50 million tons (9) and is expected to increase rapidly. In addition to the concern for proper treatment and disposal of hazardous waste produced now and in the future, there is even more concern about past disposal practices. As is widely known, a large number of uncontrolled hazardous waste sites have been discovered (9) and the number is increasing. Although these sites have been abandoned, the damage to the environment has either already occurred or can be expected to occur in the future. A survey of 204 disposal sites (15) concluded that available techniques for remedial action are not adequate for the complete cleanup of ground and surface water. Neely et al. (15) emphasized the need for innovative techniques for treatment, and effective methods for the complete cleanup of uncontrolled hazardous waste sites.

OBJECTIVES

This project has been conducted in two phases. The objectives of Phase I are:

1. To review the literature on potential effects of freezing on the level of contaminants at existing hazardous waste sites.
2. To evaluate the feasibility of using natural and artificial freezing as an alternative technique for treatment of hazardous waste.
3. To report on the technical concept for the artificial freezing process and availability of hardware.
4. To develop a cost and economic analysis of artificial freezing for hazardous waste remedial action.

TABLE 1. EXPERIMENTAL PLAN, ION TRANSPORT IN FREEZING AND FROZEN SOILS

Treatment	1	2	3	4	5
Water content	160%	160%	160%	160%	160%
Heavy metal added	yes	yes	yes	yes	yes
Freeze/thaw cycles	1	1	3	1	0
Drainage	no	yes	yes	yes	yes
Drainage period	-	2 days	2 days	extended	extended
Leachate chemical analysis	yes	yes	yes	yes	yes
Soil chemical analysis	frozen	thawed	thawed	thawed	thawed
Photos	yes	no	no	no	no
Thin section	yes	no	no	nc	no

The objective of Phase II of the project is to conduct laboratory studies on the effect of freezing on the fate of selected metals and organics in contaminated soils.

APPROACH

Phase I of this project was conducted by searching the literature on the use and effects of artificial and natural freezing on contaminants in uncontrolled hazardous waste sites, sea water, wastewater, sludge and soils. The search indicated that there is very little information available on the effects of freezing on the level of contaminants at uncontrolled hazardous waste sites. However, a great deal of research has been conducted on the use of freezing for dewatering sludge, for desalting sea water, and for constructing mines and other civil engineering projects. The relevant findings will be summarized in this paper and in more detail in a separate project report submitted to the USEPA.

The approach taken for Phase II was to investigate the fate of selected heavy metals and organics in soils upon freezing. The selected metals were Cu, Zn, Cd and Ni, and the organics included toluene, benzene, chloroform and tetrachloroethylene. The heavy metals experiment was conducted separately from the organics experiment to avoid possible interactions and the need for different techniques for sample handling. The effect of freeze/thaw cycles on the quantity and quality of leachate was investigated. Table 1 shows the experimental design for the metals portion of the study. The setup for the organics was similar to that of the metals except that soil column 4 was subjected to two cycles of freeze/thaw and soil column 3 was subjected to five cycles of freeze/thaw. The columns used were made from plexiglass, 12.5 centimeters (cm) in diameter and 80 cm in height, and were filled with either metal-spiked or organic-spiked soils. Visual observations were made during the freeze/thaw cycles and the volumes of leachate were recorded with time. Subsamples of leachate were analyzed for the desired metals or organics. Metals were analyzed by atomic absorption spectrophotometer or by inductively coupled plasma (ICP). The organics were determined with a gas chromatography-mass spectrophotometer (HP 5992 GC-MS) equipped wiht a purge trap sampler (HP 7675A). Deuterobenzene (C_6D_6) in tetraglyme was added to each sample as an internal standard just prior to purging. The rate of frost penetration was recorded with time using thermocouples and a data logger. At the conclusion of the experiments, photographs and thin sections of the frozen column were made to document the effects of freeze/thaw on soil structure and the formation of ice lenses. Subsamples of the soils with depth were chemically analyzed for metals or organics.

PROBLEMS ENCOUNTERED

Some difficulties in the extraction and the analysis of organics were encountered which made it necessary to improve on current analytical methods. The improved techniques will be published elsewhere (Jenkins, in preparation).

RESULTS

The literature search indicated that natural and artificial freezing has been used for over 100 years for mining (4), for desalting of sea water (11,12,18,20), for

TABLE 2. NORTH AMERICAN MINE SHAFT GROUND FREEZING (4)

Mine	Location	Year	Shaft dimen., m	Shaft depth, m	Freeze depth, m
Chapin Mining Co.	Iron Mt., Michigan	1888	4x5		30
PCA	Carlsbad, NM	1952	4.6	233	107
PCA	Sask., Canada	1955	4.9	1,051	914
Myles Salt Co.	Louisiana		4.9	228	761
International Min.	Canada	1957	5.5	1,030	363-437
Duval	Carlsbad, N.M.	1963	4.26	305	128
International Min.	Canada	1963	5.64	1,021	468
Allan Potash Mines	Canada	1964	4.9	1,089	625
Alwinsal Potash	Canada	1964	5.5	1,004	527
Noranda Mines Ltd.	Canada	1965	4.9	1,052	591
Cominco Ltd.	Canada	1965	5.64	1,089	684
PCA	Sask., Canada	1967	5.5	1,068	548
PCS	Canada	1967	4.9	1,021	461
PCS	Canada	1967	4.9	1,000	469
Cargill	Louisiana	1969	4.9	378	70
Cargill	Louisiana	1973	4.9	478	76
PCS	Canada	1974	4.26	1,004	527
Amax Coal Co.	Illinois	1974	6.09	228	38
Diamond Crystal	Louisiana	1975	2.44	470	70
Island Creek Co.	Kentucky	1975	6.09	122	54
Morton Salt Co.	Louisiana	1977	5.5	381	64
Peabody Coal Co.	Illinois	1978	6.09	122	53
Selco Mining Corp.	Quebec, Canada	1979	3.65	61	61
White County C. Corp.	Illinois	1979	6	335	41
Domtar	Goderich, Ontario	1980	6.09	518	61
Les Mine Salin	Mad. Is., Canada	1980	6.70	122	43
Turris Coal Co.	Illinois	1981	7.3	87	64
Asarco Exploration	Timmons, Ontario	1981	3.65	174	43

treating wastewater and sludges (2,3,5,10), for supporting large buildings during excavation and construction, and for tunnel construction (4,6,8,13,21). Braun and Nash's (4) information on the use of artificial ground freezing for building mine shafts in North America, presented in Table 2, indicates that artificial ground freezing can be extended to a depth of more than 900 m below the ground surface and that the hardware is currently available.

Artificial and natural freezing in desalting projects works because, when the temperature of the saline solution is gradually lowered below 0°C, ice nucleation starts and ions are rejected. As the temperature is further lowered, a large mass of ice is formed and a concentrated brine is left behind. The ice is then washed to remove external salt and then it is removed to another site and allowed to melt, forming fresh water. Alternatively, the brine is pumped out and the ice is left to melt. The same principle can be used for treatment of industrial effluent. However, in this case we will be interested in the concentrated solutions for reuse, recovery or further treatment. The gradual lowering of temperature is very important because, if the temperature drops fast, ions or salts are entrapped between the ice particles and the separation process will not be complete.

The main purpose of sludge treatment by freezing is for dewatering or volume reduction. Alum $[Al_2(SO_4)_3]$ enhances coagulation and improves the physical properties of the sludge. It also improves the quality of the separated water, as heavy metals and nutrients such as phosphate will precipitate with Al ions. Some systems, such as those in Nekoosa and Port Edwards, Wisconsin (16), and Burlington, Canada (17), operate on a large scale. Both systems are using natural freezing. The cost of energy was a drawback in some projects; however, this method should be evaluated in the case of hazardous waste.

Chamberlain and Blouin (7) investigated the use of freezing for dewatering polluted dredged material, and they found that freezing and thawing were effective. Freezing affected the consolidation of dredge material by 1) aggregating and restructuring soil particles, 2) densifying aggregates, and 3) segregating ice.

Ground freezing and its effects have been extensively studied in the laboratory. Anderson and Morgenstern (1) summarized the many effects of freezing on soil physical and chemical properties as they relate to engineering. The phenomenon of frost heave is probably the most widely recognized effect. Frost heave results from the growth of ice lenses in soils during freezing, which causes an expansion of the soil mass and a subsequent rise of the soil surface above the freezing zone. Ice lens growth requires water movement to the lenses from the unfrozen soil beneath.

Frost heave should be a concern in uncontrolled hazardous waste sites where frost penetration reaches the depth where waste materials have been buried. For example, soil-covered asbestos, or even buried barrels containing chemicals, could slowly move upward as a result of frost heaving. Properly designed soil covers must be deep enough so that frost penetration cannot reach the hazardous waste.

Another important finding is that some amount of unfrozen water is always present in frozen soils as a thin film around the particles. The amount of the unfrozen water depends on factors related to soil physical and chemical characteristics and temperature. This is important in hazardous waste sites because chemicals can move even in frozen soils as concentrated solutions in the unfrozen water films. Also, in calculating the freezing point of contaminated soils, engineers must remember that the chemicals could depress the freezing point of the soil solution.

The successful use of artificial ground freezing in many large-scale engineering projects (Table 2) indicates that the technology and the necessary hardware are available. Artificial ground freezing uses the Poetsch method. Briefly, this method involves spacing vertical drill holes with concentric refrigeration lines along the desired freezing line. A header or manifold system provides coolant to an interior pipe, with the return line being the outer casing (Fig. 1). A self-contained refrigeration system pumps coolant around the freezing loop. Temperature-measuring instrumentation is appropriately placed to monitor the progress of the freeze front. The parameters controlling the energy and time needed to freeze a soil include soil thermal properties, refrigeration line spacing, soil water content, initial temperature of soil and atmosphere, and temperature of coolant.

Basically, two techniques for coolant circulation have been used. The first involves drilling holes in the ground around the area to be excavated, and circulating brine solution or other coolants at low temperature as discussed above (Fig. 1a). This technique cools the soil slowly and allows for soil water or groundwater to migrate and to form large ice lenses. The

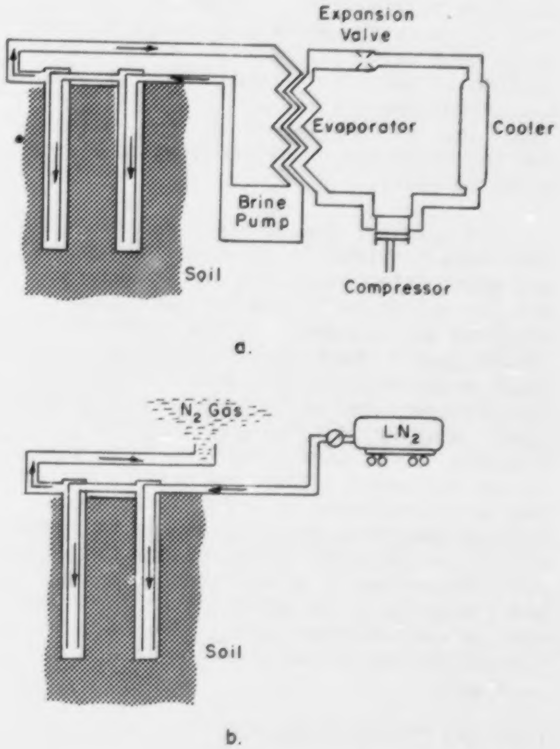

Figure 1. Artificial ground freezing by brine (a) and liquid nitrogen (b).

advantages of this technique are its low initial cost and maintenance cost, and the regularly shaped walls it forms. Its disadvantages are the length of time it takes to reach the desired low temperature and the fact that the lowest temperature achievable is only -55°C. The second technique uses liquid nitrogen (LN_2) for cooling (Fig. 1b). The use of LN_2 has the advantage of reaching much lower temperatures (-196°C) in a much shorter time compared with the brine cooling technique. Its disadvantages, however, are its high cost and the irregularly shaped walls it forms. Both techniques may be used in uncontrolled hazardous waste sites, depending on the situation and purpose.

For example, the brine cooling technique may be used for separation of solids from liquids, forcing the contaminants to move in soil solution in a desired direction, and in situations where time is not an important factor. The use of LN_2 will be necessary if one wants to entrap contaminants in the soil mass, when time is a critical factor, and when a very low temperature is required. A very low temperature is necessary where high concentrations of soluble salts or chemicals are present. In some situations, a combination of the two techniques - using LN_2 initially to reach the low temperature and then brine cooling to maintain the freezing condition - may be ideal.

In summary, ground freezing may have a detrimental effect on uncontrolled hazardous waste sites because of frost heaving and ion migration in the leachate upon freezing and thawing. In the northern United States where many freeze/thaw cycles occur every winter, the impact on contaminant movement and soil cover must be evaluated. However, natural and artifical freezing can be used beneficially in some situations under proper engineering design. The beneficial uses of freezing at uncontrolled hazardous waste sites are to immobilize contaminants and to change the physical properties of sludge and sediments, and currently it is being tested for treatment of contaminated soils. The hardware for artificial ground freezing is now available.

COST AND ECONOMIC ANALYSIS

Artificial ground freezing for hazardous waste containment is a proposed alternative to the traditional slurry wall or grout curtain barrier techniques for immediate action to halt contaminant movement and for treatment of soils and sediments. The parameters quantified for the cost analysis of ground freezing include the thermal properties of soils, refrigeration line spacing, equipment mobilization and freezing time constraints. Sullivan et al. (19) developed a computer program and a procedure for analyzing the cost of using artificial ground freezing at uncontrolled hazardous waste sites. The thermal data required for the cost analysis are obtained primarily from soil texture, water content and temperature measurements. The specific heat of soils depends on the water content because the volumetric heat capacity ratio of water to most dry soils is about 5. The thermal conductivity of coarse-grained soils is significantly higher than that of fine-grained soils. Increasing the water content of soils decreases the thermal conductivity. A water-saturated soil system is desirable for formation of an impermeable frozen barrier. Lunardini (14) provides extensive data relating soil physical properties to soil thermal properties. Therefore, soil thermal properties can be predicted from soil physical properties.

The time constraint for forming the frozen wall plays a critical role in some situations where immediate action is required. In these cases, the drilled holes should be closer to each other and liquid nitrogen should be used. The cost associated with this process is high. If time is not important, such as in cases where treatment of contaminated soils or sediment is desired, drilling spacing can be large and reusable refrigerant used; the cost would be much less. The cost for equipment mobilization and site preparation depends on the site location, proximity to highways and contractors, and the nature of the site.

The following hypothetical case gives an overview of the calculations necessary to obtain an optimum design for a ground freezing project and to compare the cost with other alternatives. Assume a chemical spill occurred as a result of a truck accident or a derailed train car (Fig. 2). Surrounding towns impose a 30-day time constraint on the chemical and transportation companies for containment of the spill to avoid contamination of a nearby water resource. A preliminary 1-week field inves-

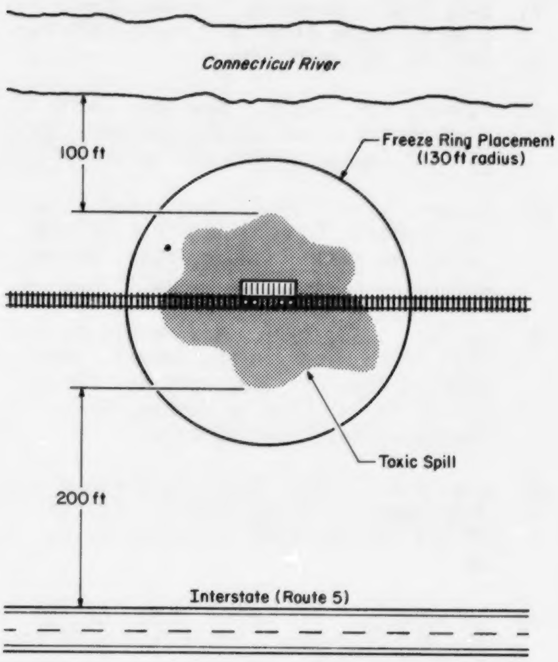

Figure 2. A hypothetical chemical spill (after Sullivan et al. (19)).

tigation to define the problem and to obtain data on soil physical and thermal characteristics is required. Assuming that the pollutant diffuses horizontally at a rate of 1 foot (ft) per day, the frozen wall is planned as a circle with a radius of 130 ft around the spill. From the investigation, the pollutant had reached a 15-ft depth. Figure 3 summarizes the cost analysis data obtained.

The optimum cost design (least cost) is about $70,000 (Fig. 3). This design calls for 3.8-ft drill spacing with a 12-day freezing time. If time is not sufficient to freeze the soil, drill spacing must be decreased to 3.7 ft and the cost of drilling will be higher. From Figure 3, the cost of fuel is not as sensitive as time, drilling equipment rental and drilling expenses. The time and drilling cost (dependent on drilling spacing) are the most sensitive parameters in the cost of artificially freezing the soils. The more time that is available, the lower the drilling cost.

Although this is not an actual case, it provides the reader with an idea of how the program works and what parameters should be considered in planning to artificially freeze the ground. For details on the model and the case discussed, the reader should consult Sullivan et al. (19). In this example, ground freezing is used for immediate action and only as a temporary treatment because of the maintenance cost. The added features of low noise and minimal environmental disturbance when using ground freezing may be desirable in urban areas. The data needed to calculate the required thermal properties of soils for technical and economical analysis are routinely obtained during the geotechnical and hydrological site investigations. Consequently, there are no additional site examination costs.

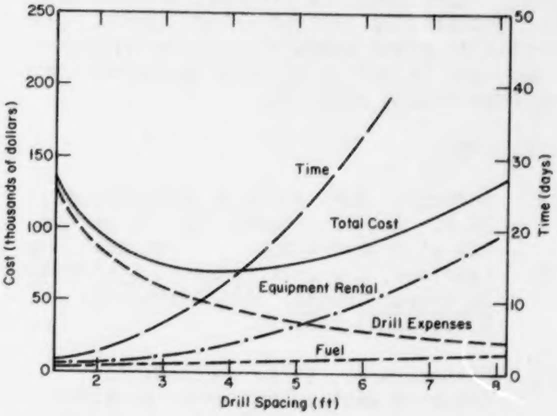

Figure 3. Cost of artificial ground freezing as a function of time and drill spacing (19).

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OPERATION OF THE CENTER HILL PILOT PLANT AND SOILS LABORATORY FOR SUPERFUND AND RCRA RESEARCH PROJECTS

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ABSTRACT

The Center Hill Pilot Plant and Soils Laboratory is operated under government contract with the University of Cincinnati and provides the U.S. EPA Hazardous Waste Engineering Research Laboratory (HWERL) with technical support for various Superfund and RCRA research projects.

A poster and slide presentation will present the capabilities of the facility and the general conduct of current research projects.

The facility consists of an engineering laboratory complex to support geoscientific and geotechnical research with emphasis on soil/chemical interactions.

A collection of "one-pager" handouts on each project will be available. Many of the U.C. principal investigators and EPA technical project monitors will be attending the seminar and be available to discuss their projects.

Capabilities of the University of Cincinnati at the Center Hill Facility include:

- Geotechnical and geochemical laboratories to perform all conventional soil mechanics lab tests and those special tests which deal with hazardous waste.
- A special permeability laboratory where studies on soils and waste material are performed to determine hydrological and chemical resistance characteristics.
- A geoscientific laboratory where pilot scale and bench scale studies for the treatment of hazardous waste and contaminated soils dealing with biological and other special treatment processes are conducted.
- A high bay area where pilot scale testing equipment for studies on the performance and treatment of various waste remedial techniques are conducted.
- A wet chemistry laboratory for conventional lab studies for treatment processes.
- The facilities for conducting studies of gas permeation through flexible membrane liners.

RECLAMATION AND REUSE OF CONTAMINATED LAND IN THE UNITED STATES, ENGLAND, AND WALES

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ABSTRACT

This poster will provide information on selected case studies illustrating the reclamation and reuse of land contaminated with hazardous wastes. Examples presented will be taken from a larger set of case studies compiled by the Research Triangle Institute as part of a study to investigate issues surrounding the reclamation and reuse of contaminated land including uncontrolled hazardous waste sites.

Examples of redeveloped sites in the U.S. and the U.K. have been documented through record searches and site visits. The following sites are included.

Frankford Arsenal, Philadelphia, PA--cleanup and redevelopment of 160-year old arsenal for reuse as downtown commercial/industrial center.

Hercules Properties, Hercules, CA--cleanup and redevelopment of 70-year old explosives manufacturing property into commercial office park and convention center.

Kapkowski Road Site, Elizabeth, NJ--cleanup of uncontrolled waste dump contaminated with PCB-laden oil and redevelopment as industrial park.

Thamesmead, London, England--reclamation of properties associated with 300-year old Woolwick Arsenal for redevelopment as housing for 60,000 people.

Beaumont Leys, Leicester, England--cleanup and redevelopment of former sewage works and 200-acre sewage farm for redevelopment as a new town.

Ravenfield Tip, South Yorkshire, England--cleanup of former uncontrolled hazardous waste site and subsequent landscaping as football field and public open space.

Where meaningful, comparisons will be made between the practices of the U.S. and the U.K. with respect to reclamation and reuse of contaminated lands.

COMPUTER ANALYSIS OF HAZARDOUS WASTE DIKES

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ABSTRACT

The objective of the study was to develop a "user-friendly", interactive computer program for the geotechnical analysis of dikes used around hazardous waste sites. The program was designed to guide a permit reviewer along the steps of the analysis of an earth dike, considering slope stability, bearing capacity, settlement, liquefaction, piping, hydraulic flow and pressure conditions. The program is to be used on the IBM-PC/XT microcomputer.

In addition to the input and output subroutines, and the controlling main program, the following subroutines have been developed:

- A slope stability and bearing capacity analysis program. For this, the REAME program by Dr. Huang of the University of Kentucky was adapted and rendered user-friendly. This program is based on the Modified Bishop Method and, in its adapted form, it will perform an automatic search for the minimum factor of safety with respect to slope stability and bearing capacity;
- A settlement subroutine analysis program to calculate the maximum settlement of a dike and also the differential settlements between the centerline, crestline and toe of a dike. The subroutine handles the cases of both normally-consolidated and overconsolidated sub-layers;
- Hydraulic subroutine programs to analyze the flow of water or leachate through and under a dike to calculate hydraulic pressures and to check for potential liquefaction and piping in the dike system.

A User's Manual was also prepared to facilitate the use of the program.

EVALUATION OF STABILIZED DIOXIN CONTAMINATED SOILS

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ABSTRACT

Asphalt and portland cement are being tested as stabilizers to reduce the release of dioxin from three contaminated soils from the St. Louis, Missouri, area. The soils include a sandy loam from the Piazza Road site, a sandy silty loam from the the Minker site, and a silty loam from the Sontag Road site. Standard mix design methods for preparing soil asphalt using emulsified asphalt and cutback asphalt, and for soil cement, are being followed to prepare test specimens meeting highway industry criteria for strength and weathering performance. Cement contents ranging from 7 to 12 percent have produced acceptable specimens for the three soils. The initially designed emulsified asphalt specimens did not meet acceptance criteria at any asphalt concentration, probably due to the high soil fines content, so a lime treatment is being tested to agglomerate the fines before asphalt stabilization.

Once acceptable soil cement and soil asphalt mixes are designed, fresh test specimens will be prepared and subjected sequentially to repeated wetting and drying cycles, then to repeated freezing and thawing cycles, and then to prolonged leaching with water according to a modified solid waste leaching protocol. Dioxin is known to have very low water solubility, so movement of dissolved dioxin into the environment is not expected to be a significant factor. But the movement of dioxin into the environment by erosion and the suspension in surface runoff of dioxin contaminated clay and fine silt particles can potentially be a significant release mechanism. Therefore, both the leachate and soil fines suspended in the leachate will be analyzed for 2,3,7,8-tetraclorodibenzodioxin (TCDD), the most toxic dioxin isomer, to assess the effectiveness of cement and asphalt stabilization methods for preventing the escape of dioxin into the environment.

TWO AUTOMATED SYSTEMS FOR ON-SCENE COORDINATORS: THE COUNTERMEASURE SELECTION SYSTEM AND THE CASE HISTORY FILE OF RESPONSES TO HAZARDOUS SUBSTANCES RELEASES

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ABSTRACT

Two automated data bases have been developed for the Releases Control Branch, Land Pollution Control Division, Hazardous Waste Engineering Research Laboratory, USEPA by Science Applications International Corporation (SAIC). They are intended to be used by On-Scene Coordinators (OSC) and their staffs in planning and implementing spill cleanups and remedial response actions. They can be accessed directly by computer terminal from the response site if desired.

The Countermeasure Selection System provides On-Scene Coordinators (OSC) with a computerized database of information to help them select appropriate countermeasures when responding to hazardous spills. The system is based on the EPA "Manual of Countermeasures for Hazardous Substance Release", a compendium of information on the hazards and behavior of chemicals when spilled and potential spill countermeasures.

The system guides the user along a decisionmaking path by providing information focussed on the incident at hand. The information in the database consists of data arranged in three tables:

- Characteristics of the hazardous substances spilled
- Applicable countermeasures for the chemical class of the spilled substance
- Limitations and requirements of applicable countermeasures

For example, an OSC might want to determine applicable countermeasures for an incident involving a railroad tank car spill of acrylonitrile in a flood plain. The OSC would look up the chemical in the first table to determine its characteristics; that leads to the second table, which identifies technically feasible countermeasures. The third table presents the limitations and special requirements of the countermeasures and gives references to paragraphs in the manual having further information to help the OSC reach a final decision.

The Case History File provides a database of information pertaining to past responses to releases of hazardous substances. The File is a computerized system that helps an OSC's staff determine:

- if there have been previous incidents like this one,
- how they were handled,
- how successful the effort was,
- how much it cost.

Previous incidents can be characterized and accessed by:

- date
- location
- hazardous material involved

- interacting substances
- geographical and geo-hydrological characteristics
- affected area, population, and resources
- containment actions
- cleanup actions
- treatment technologies used
- other factors.

The File also contains narratives with additional information on each entry, including the OSC's rationale for taking a particular course of action. Cost information is included to help gauge the feasibility of a given response action.

For example, an OSC might want to find information for a case involving a railroad spill of acrylonitrile in a flood plain. Then he might want to read the text to see what unique problems might be encountered or what containment methods were used and how well they worked.

The system also provides an interactive procedure for entering after-action reports on new incidents. This helps ensure the timely entry of new data and improves the currency of the data base.

The Countermeasure Selection System and the Case History File are two automated systems for OSC use that are presently being developed for the Releases Control Branch, Land Pollution Control Division, Hazardous Waste Engineering Research Laboratory.

VOLATILE ORGANIC EMISSIONS REDUCTIONS FROM SURFACE IMPOUNDMENTS BY THE USE OF WIND FENCES AND WIND BARRIERS

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ABSTRACT

Quantities of hazardous waste materials in liquid, sludge or solid form can be disposed on or near the soil surface. A common disposal operation uses surface impoundments for aqueous waste. Proper design and operation procedures can result in significantly reduced emissions of hazardous volatile species to the surrounding air.

Surface impoundments usually consist of shallow earthen basins. Aqueous and non-aqueous waste are placed in these open top basins for treatment of selected waste or for storage. Volatiles may escape to the air boundary layer but significant reduction in emissions rate is possible by the use of perimeter fences and network fences. Desorption data was obtained on a pilot scale apparatus using ethyl ether as the model chemical. Forty percent reduction in emission of ethyl ether was achieved with this liquid-phase controlled volatile chemical using solid perimeter fences. Seventy eight percent reduction was achieved with solid fence networks.

INTRODUCTION

A surface impoundment is typically a large, open-top vessel of earthen construction containing aqueous waste. These vessels can also contain a floating hydrocarbon layer on the surface and/or a semi-solid sediment on the bottom. One important characteristic is that it has a liquid interface in direct contact with the atmosphere. Hazardous substances including chlorinated solvents, hydrocarbon solvents, pesticide waste, inorganic and other classes of volatile chemical compounds are contained in some of these impoundments. These are usually surface impoundments used for managing hazardous waste.

Uses of surface impoundments for hazardous waste management include: biochemical treatment of the organic substances, chemical treatment of organics and inorganics (i.e., neutralization), holding (storage) prior to performing other treatment operations and water evaporation for liquid volume reduction. While these uses are occurring, volatile chemical species in solution of the aqueous phase can evaporate to the air.

There are problems associated with the emission of volatile organic chemical from surface impoundments. The loss of solvents in large quantities can modify the general ambient air quality of the nearby air mass from the point of view of photochemical oxidants content. Odors, associated with some selected organic and inorganic compounds, are a common impact on local environment near residential areas (Wilson, et. al., 1980). The emission of specific hazardous chemicals such as benzene (Thibodeaux, et. al.) can result in increased levels of exposure (and health risk), for both workers and nearby residents.

Control methods for volatiles have been proposed and there are several possible techniques that can be used to reduce emissions. These control methods fall into three general categories: covers, fences and adsorbents. Covers involve the placement of a barrier at the water-air interface. Oil (slicks) layers, plastic films, mats of floating materials and foams have been proposed. Fences, both perimeter and surface networks, have been proposed to partially nullify the wind enhanced volatilization. On-bottom or suspended adsorbents (liquid droplets and solid particles) have been proposed as a means of retarding the evaporation by reducing the effective concentration in the water (Cudahy and Sandifer, 1980). All three techniques inhibit evaporation allowing for some treatment to be effective.

This manuscript presents a brief description of the chemodynamics mechanisms and mosels for volatile chemical emission from surface impoundments. Analysis of the rate equation suggests ways of controlling volatile emissions. Finally experimental results of emission reduction schemes using wind barriers in pilot-scale equipment are presented. Preliminary design equations are proposed.

CHEMODYNAMIC MECHANISMS OF VOLATILIZATION

The general concepts to explain emission of volatile chemicals from aqueous solutions in surface impoundment type operations were developed by Thibodeaux and Parker in 1974 (Thibodeaux, 1979). A brief review of these mechanisms and models is needed.

Volatilization will be considered to be occurring with the chemical in aqueous solution at a concentration of C_w, g/cm^3. If the overlying air mass contains the volatile chemical species at concentration C_A, g/cm^3, and C_A/H is less than C_w, than the emission flux rate from the water surface is

$$N = K\ (C_w - C_A/H) \qquad (1)$$

with N in g/cm^2·s. H is the Henry's constant for the volatile species in cm^3 H_2O/cm^3 air and K is the overall transport coefficient in cm/s. This coefficient is commonly expressed in terms of two resistances:

$$1/K = 1/k_1 + 1/k_g H \qquad (2)$$

where the $1/k_1$ term accounts for the resistance on the liquid-side of the interface and $1/k_g H$ is the gas-side resistance. For most chemicals of a hazardous nature $k_g H >> k_1$ and the liquid phase controls the volatilization process. Each of these terms must account for both mechanical and nature stimulated processes.

Wind constitutes a natural process that has a large influence on the magnitude of k_1 and k_g. Generally k_1 increases with the square of the wind velocity and k_g increases proportional to the wind velocity. In the absences of wind the temperature difference between water and air is the important independent variable. If the water is warmer than air, both k_1 and k_g are a function of ΔT to the 1/4 power. Mechanical surface aerators, installed for increasing the rate of molecular oxygen transfer to the water, also have a dramatic effect on k_1 and to a lesser extent on k_g.

IMPLICATIONS FOR VOLATILE EMISSION CONTROLS

Laboratory and field studies indicate that the above mechanisms are operative in surface impoundments and that Equations 1 and 2 represent adequate models for quantifying volatile chemical emission rates. This being the case, then an analysis of the model equations and concepts may lead to methods by which the emission rate may be reduced.

Volatilization is a well known first order process as indicated by Eq. 1. Reducing the concentration of the volatile species in the water will then reduce the emission rate a proportional amount. A reduction in the overall transport coefficient can bring about the same effect. For a given concentration level in the water, reducing K provides the most practical approach to reducing the emission rate. Means of adding additional resistances in series or by increasing k_l will add to the overall resistance. This can be seen by inspecting Eq. 2.

Adding additional resistances can be performed by placing material at the air-water interface. This is the technique of oil films, plastic films, floating mats, etc. mentioned above. This process has some disadvantages that include reduced oxygen transfer, high maintenance, incomplete coverage (oil films) and additional volatile source (oil evaporation).

The use of fences to reduce the effect of the wind on both k_1 and k_g is a potential control method that increases the overall resistance. Parker (1984), employing a theoretical model developed by Levich for the influence of protrusions in a fluid stream in reducing mass transport, along with Chilton-Colburn analogy theory, developed an expression that related k_1 to the protrusion height:

$$k_1 = \text{constant } h^{-\frac{1}{2}} \tag{3}$$

where h is the vertical fence height in cm. Advantages of fences are low installation and maintenance costs.

PILOT-SCALE EXPERIMENTS WITH FENCES

Parker (1984) performed a series of pilot-scale simulations in a wind tunnel water tank (WWT) to investigate the effectiveness of fences in reducing volatile chemical emissions from water bodies. The wind tunnel was 6 meters long and 0.6m x 0.6m in air flow area. Underneath was a water tank 0.6 m maximum depth and 0.6 m wide. The air-water contact surface length was 2.4 m. Ethyl ether, which is a liquid-phase controlled volatile, was placed in the water tank at an initial concentration of approximately 100 mg/l. Batch evaporation experiments, involving monitoring of the decrease in ethyl ether concentration with time and extraction of the first order rate constant, yielded valves of k_1. Numerous batch experiments with various fence arrangements yielded various values of k_1. The uncontrolled k_1 was obtained without a fence or obstruction present in the WWT.

A total of 37 batch desorptions were performed with various fence arrangement around the water surface in the WWT. Two basic fence patterns were investigated. These were perimeter fences and network fences. A perimeter fence is one that encircles the impoundment at the water's edge. Network fences involve the installation of fences on (i.e., floating or piles) the water surface creating a series of uniformly spaced cells.

The simulation experiments performed in the WWT were scaled to an existing surface impoundment studied at one particular hazardous waste facility in western New York State (Thibodeaux, et al.). This surface impoundment was nearly square in plan area with sides of 186 meters and 4.23 meters in depth. Table 1 shows the relation and scaling factors between the field impoundment and the laboratory model.

The degree of emission reduction is defined by:

$$\%\ \text{Reduction} = \frac{N_{w/o} - N_w}{N_{w/o}} \times 100 \tag{4}$$

where $N_{w/o}$ and N_w is the emission rate without fences and with fences respectively. To nullify the effect of air concentration, C_A was kept at zero or at $C_A/H << C_w$ in the inlet air at all times. Also the effect of C_w can be nullified by assuming identical values both with and without fences. Substituting Eq. 1, into Eq. 4 with the above conditions, yields:

$$\%\ \text{Reduction} = \frac{k_{1,w/o} - k_{1,w}}{k_{1,w/o}} \times 100 \tag{5}$$

So it is possible to quantify the emission reduction potential of a particular fence arrangement by measuring k_1 values.

The results of experiments at wind speed of 390 cm/s appears in Table 2. These results represent an average of three to five batch experiments for each fence arrangement. The 90° angle indicates a vertical fence; the 80° angle indicates a 10° downwind tilt and 45° angle a 45° downwind tilt to the upwind fence. The porous fence as a commercial fabric sold under the trade name DUSTAMER®*. The two series of network fence studies are for fences placed 5h and 20h distance apart on the water surface, where h is the fence height.

The simulation test results represented in Table 2 include the ethyl ether desorption mass-transfer (m-t) coefficient, k_1 in cm/h, and the standard deviation (sd). The first line entry in the table is k_1 without any barriers. The percent reduction for each barrier arrangement is calculated using Eq. 5 and appears in the last column. For a vertical perimeter fence with an air speed of 390 cm/s, the volatile chemical desorption rate is reduced by 19.7% for a low (1.7 cm. model height = 1.2 m. field height) fence and 34.3% for a high (6.4 cm. model = 4.8 m. field) fence.

Results from a high fence tilted 10 degrees downwind indicates a 39.1% reduction. However, if the $k_1 \pm 2$ sd is calculated and compared for each angle there is significant overlap. This suggest that the difference in the average percent reductions are insignificant. A slight downwind tilt, as used in snow drift control, appears to have no significant effect in volatile emission control.

The last three line entries in Table 2 represent a porous perimeter fence and a network of solid fences. The $k_1 \pm 2$ sd for the no-fence case and the porous fence case do not overlap. The 78.3% reduction is therefore significant and represents a high degree of volatile emission control. Simulation results for a series of fences (i.e., networks) spaced 5x heights and 10x heights apart over the surface are also significant and the degree of volatiles control is also high. (F=fence spacing).

It is possible to combine the results of both the perimeter and network experiments by using a dimensionless scaling factor. The study of k_1 values in Table 2 suggest that spacing between fences is an important independent variable. The use of the dimensionless ratio, F/h, seems to allow consolidation of both sets of data into a single parameter. Figure 1 is a plot of the 90° solid perimeter and network fence data. A linear regression of this data yielded the following equation:

$$k_1 = 0.328. \quad (F/h)^{0.4619} \qquad (6)$$

The statistical correlation coefficient was .9513 indicating a high degree of correlation. It is also interesting to note that the experimental k_1 is proportional to h to the -0.4619 power and the theoretical model was -0.5 (see Eq. 3). By employing $k_{1,w/o} = 3.424$ in Eq. 5 it is possible to obtain a generalized equation for the design of a fence volatile control scheme. Using Eq. 6 for $k_{1,w}$ yields

$$\% \text{ Reduction}=[1 - .096\ (F/h)^{.462}]\text{x}100 \qquad (7)$$

for $1.6 < F/h < 160$. The data and Fig. 1 also suggest that very little reduction is achieved for fence spacings of 160 or greater. Therefore, % reduction = 0, $F/h > 160$ completes the design equations.

A limited number of experiments were performed at a lower wind speed of 290 cm/s with fence heights of 1.7 and 3.2 cm. This data appears in Table 3. The no-fence m-t coefficient for the low wind speed is much lower than that for the high wind speed value. Lunney (1983) observed this same effect and correlated coefficients to the square of wind speed. There is a corresponding less amount of volatile chemical emitted at the low wind speed. However, it appears that the average percent reduction is slightly higher for the low wind. Comparing data in Tables 2 and 3 indicates that the low and medium, 90°, fence heights display 40.9% and 20.3% reductions for the low wind and 19.7% and 22.8% reductions for high wind respectively. The true significance of the average percent reductions in the case of low wind is questionable considering the overlap in the $k_1 \pm 2$ sd values.

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*Mention of trade names or commercial products does not constitute endorsement or recommendation for use by U.S. EPA.

Assuming the 0.462 power relation holds for the F/h variation on emission reduction for the 290 cm/s wind it is possible to obtain a correlation similar to Eq. 7. This correlation is:

$$\% \text{ Reduction}=[1-.0836\ (F/h)^{.462}]\times 100 \quad (8)$$

for $F/h < 215$. If $F/h > 215$ the % reduction is zero. Figure 2 is a graphical representation of Equations 7 and 8.

It should be noted that the above design equations (Eq. 7 and 8) are tentative in that they are laboratory observations and involve results based on one chemical and two wind speeds. Hopefully continued research will involve other volatile chemicals, additional environmental parameters including water temperature and field testing.

CONCLUSIONS

A properly conceived and formulated mathematical model of a natural process can be a powerful tool for studying means of controlling transport rates of chemical species. In the case of surface impoundments, fences appear to be an effective volatile chemical control device in the presence of wind. a perimeter fence of solid construction (i.e., non-porous) can give up to approximately 40% volatile emission reduction. A network grid can give up to 80% reduction as can a porous perimeter fence. Fence angle does not increase the effectiveness of a solid perimeter fence as a means of emission reduction.

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Table 1
Scaling Factors for Laboratory Simulation

Parameter	Laboratory Model	Field Equivalent
wind speed	290 cm/s @ 10 cm	5.6 m/s @ 10 m
wind speed	390 cm/s @ 10 cm	8.0 m/s @ 10 m
fetch*/depth ratio	44	44
fetch*	244 cm	186 m
fence height	3.2 cm	2.4 m
fence height	1.7 cm	1.2 m
fence height	6.4 cm	4.8 m

*fetch (F) = distance wind travels over open water. In this study it is distance between fences, or fence spacing.

Table 2
Fence Control Simulation Results (V = 390 cm/s)

Parameter	h, height (cm)	mass-transfer coefficient k_1(cm/h)	standard deviation (cm/h)	% reduction
No fence	---	3.424	.0395	0.0
Solid, perimeter, 90°	1.7	2.750	.224	19.7
Solid, perimeter, 90°	3.2	2.642	.406	22.8
Solid, perimeter, 90°	6.4	2.251	.160	34.3
Solid, perimeter, 80°	6.4	2.087	.085	39.1
Solid, perimeter, 45°	6.4	2.211	.134	6.2
Porous, perimeter, 90°	6.4	.7445	.119	78.3
Solid, network 90°				
F = 5 h	1.7	.7328	.0908	78.6
F = 10 h	1.7	.7540	.0454	78.0

TABLE 3
Fence Control Simulation Results ($v = 290$ cm/s)

Parameter	h, height (cm)	mass-transfer coefficient k_1 (cm/h)	standard deviation (cm/h)	average % reduction
no fence	0.0	2.375	0.333	0.0
solid perimeter, 90°	1.7	1.403	0.460	40.9
solid, perimeter, 90°	3.2	1.893	0.673	20.3

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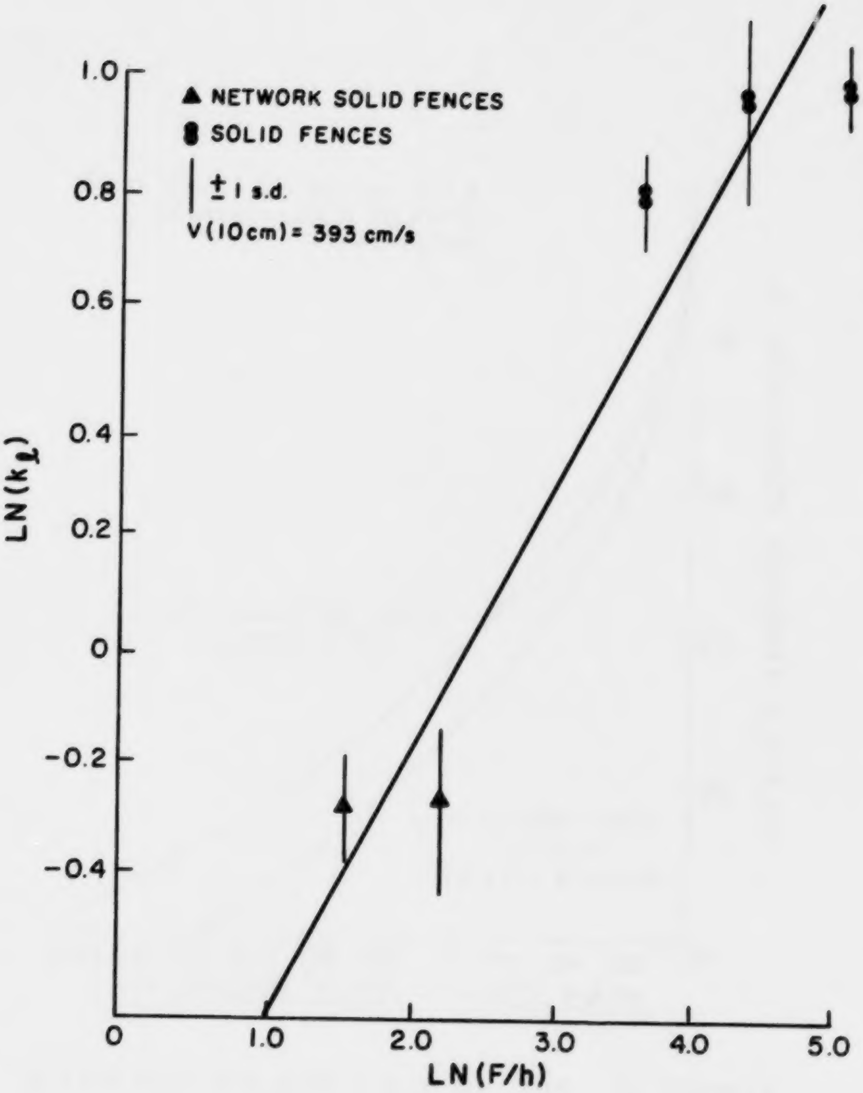

FIGURE 1. LOG PLOT OF LIQUID COEFFICIENT VERSUS (F/h)

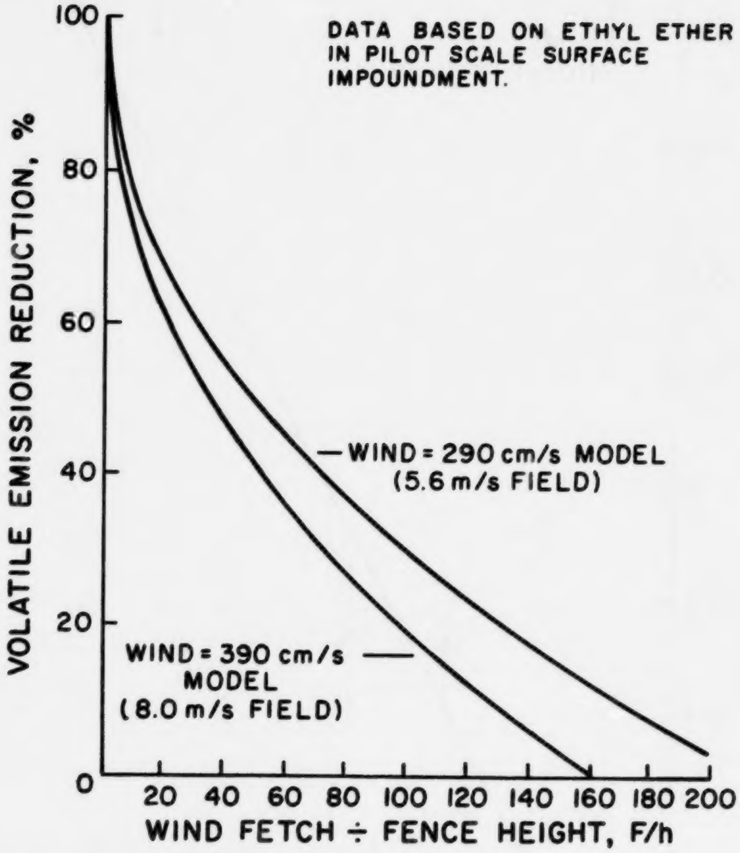

FIGURE 2. PRELIMINARY DESIGN–VOLATILE EMISSION REDUCTION BY WIND BARRIERS

INVESTIGATION OF FLOATING IMMISCIBLE LIQUIDS TO CONTROL VOLATILE ORGANIC CHEMICAL (VOC) EMISSIONS FROM SURFACE IMPOUNDMENTS

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ABSTRACT

Floating immiscible liquids as a control option for volatile organic chemical emissions from a laboratory scale waste water impoundment were investigated. Three different immiscible organic liquids were used, viz., mineral oil, octanol and lauryl alcohol. A three phase resistance-in-series model was proposed. Experiments with complete surface coverage and thick mineral oil layer showed a considerable decrease in the emission rates only for those compounds with low oil solubility. On the other hand the emission rate was less significantly reduced for chemicals with high oil solubility.

INTRODUCTION

Field measurements as evidenced by previous work [1-4] have demonstrated that significant amounts of volatile organic compounds (VOCs) are being released to the air from surface impoundments containing hazardous waste. Several control methods are being investigated which would help reduce these VOC emissions. Two such control options currently under study are:

(i) the use of floating non volatile immiscible organic liquids (under no-wind conditions) and

(ii) the modification of the turbulent boundary layer by use of wind barriers (for wind enhanced conditions).

This paper presents ongoing work on the use of floating immiscible liquids as options for controlling VOCs from surface impoundments

PURPOSE

The goal of this research project is to provide information on the feasibility of using floating immiscible liquids as a control option for VOC emissions. It has been observed that in the absence of intentional agitation, the VOC emission rate is a function of (among other factors) wind velocity [5-7]. However, in the absence of wind-induced stirring, natural convection would be a significant mechanism in the emission process when impoundment liquid is warmer than the overlying air [8,9].

APPROACH

Two approaches are being pursued in this research. One is the procurement of data on VOC emissions from a laboratory impoundment simulator containing hydrocarbons in aqueous solution covered by an immiscible liquid surface layer. The other is to develop and verify models for such emissions.

Pilot-scale Simulator: The impoundment simulator consisted of a circular tank, six feet in diameter and two feet deep. It was insulated on the bottom and sides with about 6 inches of polystyrene foam insulation. The apparatus was located indoors, in a room which was approximately 55' x 60' with a ceiling height of 14'. To further shield the apparatus from extraneous air currents, it was surrounded by an enclosure made of polyethylene film. The enclosure was approximately 16' x 16' x 8' high, open at the top. Figure 1 shows a view of the simulator and designated sample locations within the polyethylene shield. The temperature profile of the liquid as well as of the overlying air could be measured by thermocouples mounted on a standard in the center of the tank. Some additional apparatus consisted of a removable steam coil for initial heating.

An experimental determination began by filling the simulator with water and warming it to about 40-42°C using the steam coil. Four different chemicals, viz., acetone, ethyl ether, isopropanol and benzene were then separately added and stirred in for a uniform concentration. These chemicals were chosen because they represented a broad range of solubility in the mineral oil and a range of mass transfer properties, i.e., degree to which the emission might be gas or liquid phase controlled. The floating immiscible liquid was then poured onto the surface. Approximately an hour was allowed for the currents induced by mixing to subside [9]. Samples were taken at designated locations using Vacutainer tubes (supplied by VWR Scientific) and analyzed using a Tracor 565 chromotograph with flame ionization detector.

The experimental overall mass transfer coefficients were deduced from the slopes of the plots of $\ln(C/C_o)$ versus time [8] using the equation

$$\ln \frac{C}{C_o} = - \frac{K_1}{d} t \qquad (1a)$$

where d is the depth of the impoundment, C is the concentration of the organic compound in water, C_o is the initial concentration of the organic compound, and K_1 is the overall mass transfer coefficient for the organics from the aqueous phase to the atmosphere.

Three Phase Resistance Model:

A three phase resistance-in-series model was developed to explain the transport of chemicals from water to air [10]. The relevant interfaces and corresponding mass transfer coefficients are shown in Figure 2. The equation for the overall mass transfer coefficient thus derived was

$$\frac{1}{K_1} = \frac{1}{k_1} + \frac{1}{k_{oil} K_p} + \frac{1}{k_g H} \qquad (1b)$$

where

K_1 is the overall mass transfer coefficient, cm/s.

k_1 is the water phase mass transfer coefficient, cm/s.

k_{oil} is the oil phase mass transfer coefficient, cm/s

k_g is the air phase mass transfer coefficient, cm/s

K_p is the partition coefficient of the chemical between oil and water phases (dimensionless).

H is the Henry's constant (dimensionless), $\frac{\text{concentration in vapor}}{\text{concentration in solution}}$

The water and air phase coefficients can be obtained from previous correlations [8].

$$k_g = 0.50\, D_{A1}^{2/3} \left[\frac{\beta_1 g}{\nu_1}\right]^{1/3} \left[\frac{T_{oAi} - T_1}{T_{oAi}}\right]^{1/3} \qquad (2)$$

$$k_1 = 0.14 \left[g\beta_2 (T_2 - T_{owi}) \frac{D_{A2}^2}{\nu_2}\right]^{1/3} \qquad (3)$$

where

D_{A1} = diffusivity of the chemical in air (cm^2/s)

D_{A2} = diffusivity of the chemical in water (cm^2/s)

g = gravitational constant (cm/s^2)

ν_1 = kinematic viscosity of air (cm^2/s)

ν_2 = kinematic viscosity of water (cm^2/s)

β_2 = thermal expansion coefficient of water at temperature T_2 (K^{-1})

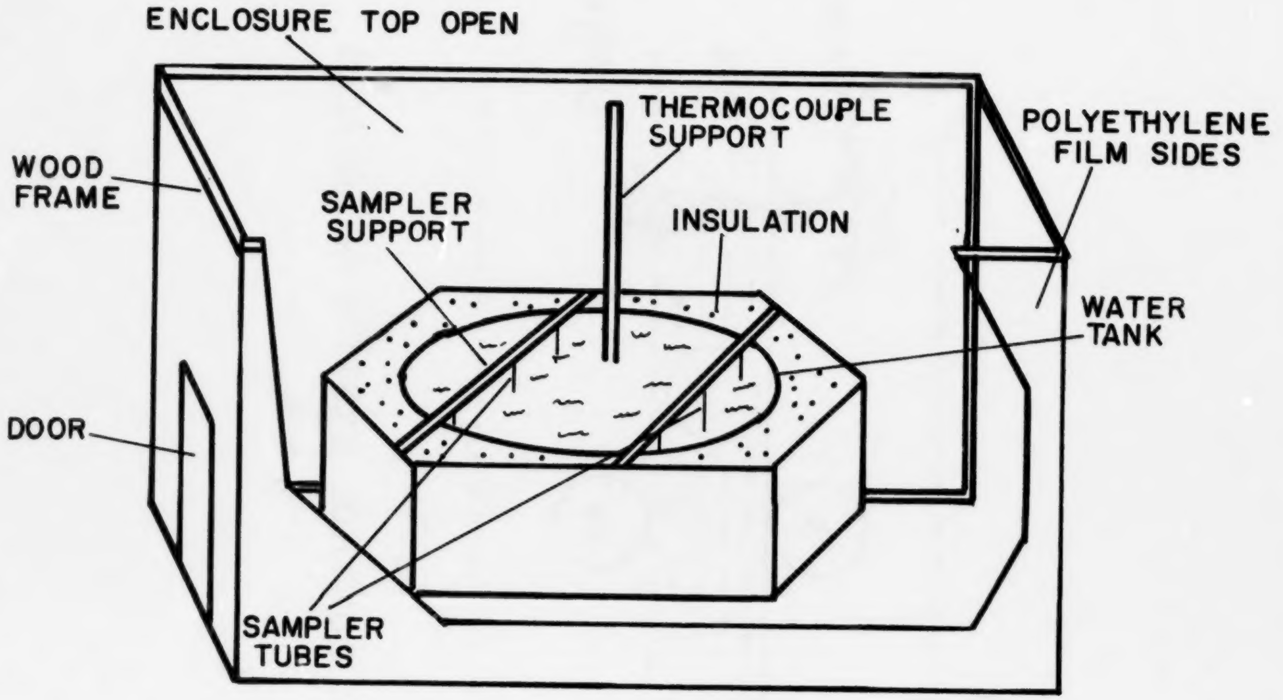
ENCLOSURE TOP OPEN
THERMOCOUPLE SUPPORT
POLYETHYLENE FILM SIDES
WOOD FRAME
SAMPLER SUPPORT
INSULATION
WATER TANK
DOOR
SAMPLER TUBES

FIG. 1. IMPOUNDMENT SIMULATOR – NO WIND
(Cut away view)

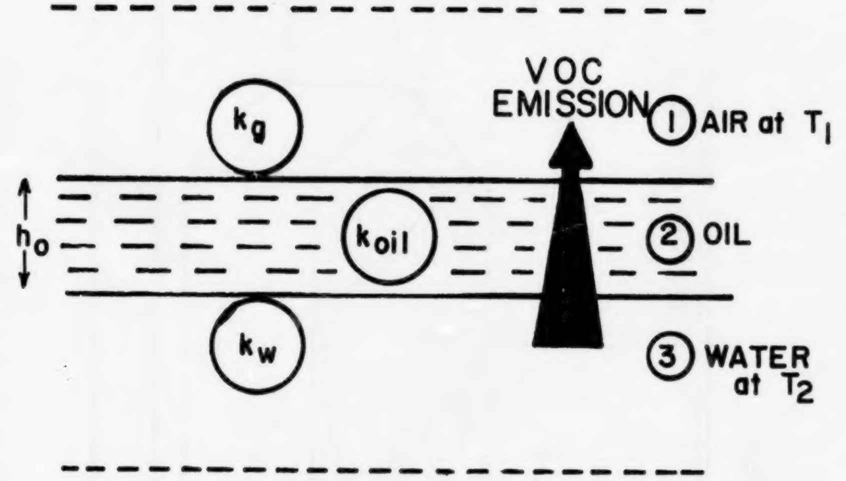
VOC EMISSION
k_g
① AIR at T_1
h_o
k_{oil}
② OIL
k_w
③ WATER at T_2

FIG 2. CONTROL SYSTEM INTERFACES
$T_2 > T_1$

β_1 = thermal expansion coefficient of air at temperature T_1 (K^{-1})

T_{oAi} = temperature at the oil-air interface (K)

T_{owi} = temperature at the oil-water interface (K)

T_1 = Air temperature (K)

T_2 = Water temperature (K)

The temperatures were measured using the thermocouples located at the various points on the thermocouple support (Figure 1).

The oil phase coefficient, k_{oil} was estimated using the simple film theory as a ratio of diffusivity of the chemical in oil to the height of the oil layer (h_o)

$$k_{oil} = \frac{D_{Ao}}{h_o} \qquad (4)$$

The oil phase diffusivity of the chemical was calculated using the Wilke-Chang correlation [7]

$$D_{Ao} = \frac{117.3E\text{-}18\ (M_{oil})^{0.5}\ T_{oil}}{\mu_{oil}\ V_a^{0.6}} \qquad (5)$$

where

M_{oil} = Molecular weight of oil (kg/mol)

T_{oil} = temperature of the oil layer (K)

μ_{oil} = viscosity of oil (kg/m.s)

V_a = Solute molal volume at normal boiling point (m^3/kmol)

The experimental value of overall mass transfer coefficient determined from experiment and evaluated by equation (1a) was then compared with the value predicted by the model from equation (1b).

RESULTS AND DISCUSSION

As of December 1984 only the mineral oil system has been completely investigated. Work on the other two systems, viz, 1-octanol and lauryl alcohol has been only partially completed and results are inconclusive. The latter two systems were chosen because the partition coefficients of the chemicals between water and 1-octanol or lauryl alcohol are readily available in the literature while for mineral oil (which is a mixture of several hydrocarbons) these measurements are not as reliable.

For the mineral oil system four chemicals (acetone, ethyl ether, benzene and isopropanol) and 5 different oil thicknesses (vis., 0.20, 0.27, 0.30, 0.38 and 0.46 cm) were investigated. Table 1 summarizes the effect of oil depth on the VOC reduction of the 4 chemicals for a water depth of 49.5 cm. Table 2 gives the experimental and predicted (by the model) mass transfer coefficients for the 4 chemicals investigated and their dependence on oil depth.

As can be observed from Table 1, the percent reduction increases slowly with the oil thickness for n-propanol, ether and benzene; whereas, for acetone the percent reduction increases rapidly as the oil layer thickens. For a 0.5 cm of oil thickness, the evaporation rates of acetone and n-propanol were reduced by 80-90%; however, with the same thickness of the oil layer it only shows about 30-40% reduction for ether and benzene.

Based on the properties of the chemicals, acetone and n-propanol are miscible in water and have very low oil solubility as is evident from their mineral oil/water distribution coefficients determined experimentally (Table 1). Because of their low concentration gradients across the oil film, acetone and n-propanol would be expected to move through the oil film very slowly after reaching the water-oil interface from the bulk water. In this case, the desorption rates of acetone and n-propanol are well controlled by the oil phase.

Table 2 shows that the experimentally determined overall coefficient, K_1, that was predicted by the model, does not agree at all for chemicals with high oil phase resistance (viz. acetone and n-propanol) while the agreement is satisfactory for those with low oil phase resistance, especially benzene. This suggests that the simple film theory prediction of k_{oil} may be in considerable error and this aspect of the model is presently being investigated. Also efforts are underway to improve estimates of oil phase diffusivities.

It has been found recently that

Table 1: Percent VOC Reduction of Four Chemicals vs Oil Thickness Under No-Wind Conditions at a Water Depth, h-49.53 cm.

Oil Thickness, cm	Temperature Ranges, °K			Percent Reduction[@]		
	Air	Water	Acetone	Ether	1-Propanol	Benzene
0.20	303-309	320-318	45.9	15.3	83.9	23.4
0.27	300-306	321-318	51.2	--	88.5	28.2
0.30	296-297	317-314	75.3	--	87.7	27.0
0.38	304-308	320-318	64.6	20.0	89.1	37.5
0.46	304-307	319-317	84.0	26.5	92.4	40.6
	Standard Deviation		7.66	1.4	0.9	1.6
Mineral oil/water distribution			0.05±0.02	3.85±0.20	0.25±0.03	2454±300

[@] Percent VOC reduction here is defined as follows (See reference 10)

$$\text{Percent Reduction} = \frac{\text{Rate of desorption without mineral oil} - \text{Rate of desorption with mineral oil}}{\text{Rate of desorption without mineral oil}} \times 100$$

Table 2: Comparison of Experimental and Model Mass Transfer Coefficients at Different Oil Thicknesses

Oil Thickness cm	Chemical	K_1 Experiment cm/hr	K_1 Model cm/hr
0.20cm	Acetone	0.97	8.97E-3
0.27cm	Acetone	0.88	6.66E-3
0.30cm	Acetone	0.44	6.66E-3
0.38cm	Acetone	0.64	4.75E-3
9.46cm	Acetone	0.29	4.06E-3
	Standard Dev.	0.14	
0.20cm	Ether	2.09	0.12
0.38cm	Ether	1.99	0.06
0.46cm	Ether	1.81	0.05
	Standard Dev.	0.04	
0.20cm	n-Propanol	0.35	8.97E-3
0.27cm	n-Propanol	0.25	6.66E-3
0.30cm	n-Propanol	0.27	6.66E-3
0.38cm	n-Propanol	0.24	4.75E-3
0.46cm	n-Propanol	0.17	4.06E-3
	Standard Dev.	0.02	
0.20cm	Benzene	1.48	1.07
0.27cm	Benzene	1.39	1.00
0.30cm	Benzene	1.41	0.94
0.38cm	Benzene	1.21	0.86
0.46cm	Benzene	1.15	0.73
	Standard Dev.	0.08	

partial coverage on the surface and formation of lenses (a thick layer of organic liquid whose shape is constrained by the forces of gravity and interfacial tensions) sometimes leads to enhanced VOC emissions. Though work on both octanol and lauryl alcohol are not far enough along to report results, there is evidence that some of those observations are due to increased circulation of the lenses on the water surface resulting from what are called "thermocapillary" effects (surface tension induced motion of lenses on the surface arising from thermal gradients in the solution, ref. 11). This explanation is only tentative and remains to be verified.

Experiments thus far indicate that the behavior of the oil vis-a-vis the volatile is of considerable significance in selecting a floating immiscible liquid as a control barrier in VOC emissions work.

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LEACHING POTENTIAL OF 2,3,7,8-TCDD IN CONTAMINATED SOILS

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ABSTRACT

Objectives

Ten soils from sites contaminated with 2,3,7,8-Tetrachlorodibenzo-p-dioxin (TCDD) were investigated to determine whether TCDD could move in water percolating through soil and to determine factors controlling potential migration. These objectives were accomplished by using batch extractions of disturbed soil samples with water and by passing water through intact core samples. Soils and extracts were analyzed for TCDD.

TCDD in Soils and Leachates

TCDD content in soil (U.S. EPA Region VII protocol) ranged from 8 ppb to 26,300 ppb (ng/g); only one of the ten soils contained more than 2,440 ppb. For the batch extractions, TCDD concentrations in solution ranged from 0.1 ppt to 56 ppt (ng/L); 79% of the values were 2.0 ppt or less. For the intact cores, TCDD concentrations in leachate ranged from 0.1 to 14 ppt; 81% of the values were 2.0 ppt or less.

Partition Coefficients

Values for soil partition coefficients (K_p and K_{OC}) were large, indicating that TCDD will be only very slightly soluble and that only small amounts will be released into water in contact with the contaminated soils used in this study. K_p ranged from 0.3×10^5 ml/g to 130×10^5 ml/g with a mean of 18×10^5 ml/g. Mean log K_{OC} for the two batch methods and the intact cores were 7.39, 7.58, and 7.55; these values were within one standard deviation of a value of 6.95 predicted from solubility data.

Effect of Co-Contaminants

TCDD concentrations in soil and in water extracts were regressed against soil physical and chemical properties to determine which, if any, were related to TCDD solubility and could be used to guide further research. Strong correlation was not obtained between clay and total organic matter content and TCDD solubility. Instead, both the amount of TCDD in the soil and the amount of TCDD released in a water extract were strongly related to the total solvent-extractable organic content in soil and the halogenated semi-volatiles in this solvent-extractable fraction. The solvent-extractable content of background (uncontaminated) soils was much lower than the contaminated soils. These results suggest that co-contaminants were in the wastes along with the TCDD and that these materials have a major role in controlling the solubility and movement of TCDD in contaminated soils.

Predictive Modeling

A solute transport model was used to predict TCDD movement rates in water percolating through soil. Using worst-case conditions (continuous saturated water flow in the soil and hydraulic conductivity of 10^{-4} cm/sec) and mean soil physical properties from the intact cores, predicted movement rates ranged from 4 yr/cm to 5367 yr/cm depending on the value of K_p and the relative TCDD concentration (C/C_o) that was used. These predicted worst-case rates confirm that TCDD is essentially immobile in water percolating through contaminated soils used in this study. The relationship between the levels of co-contaminants and TCDD solubility in soils suggests that TCDD may be more soluble and mobile in soils containing higher levels of organic co-contaminants than those in this study.

INTRODUCTION

The environmental persistence and toxicity of 2,3,7,8-Tetrachlorodibenzo-p-dioxin (TCDD) has created concern on a national level for the cleanup and safe disposal of contaminated soils. Several investigations using clean soils spiked with TCDD have documented the extreme immobility of TCDD in soils (Kearney et al., 1973; Bartleson, et al., 1975; Bolton, 1978). However, there have been no investigations of the solubility or mobility of TCDD in field soils contaminated with other wastes in addition to TCDD. This study was undertaken to assess the potential mobility of TCDD in such soils.

APPROACH

The dissolution of TCDD from soil and potential mobility of TCDD in water percolating through soil was determined either by mixing water with contaminated soils in batch studies or by passing water through intact core samples of contaminated soils. The concentration of TCDD in the soil and in the aqueous extract (soil leachate) was measured; these values were regressed against soil physical and chemical properties to identify mechanisms controlling TCDD solubility in the soils.

Bulk soil samples were collected by excavating soil to a depth of approximately 15 cm. These samples were air-dried and sieved to pass a 2 mm opening. Intact soil cores were collected from the same depths and locations by driving a 10 x 10 cm steel cylinder into the soil. The soil-filled cylinders were excavated, placed in plastic bags, and sealed in a 2-quart steel can.

Samples from soils contaminated by oil-spraying and industrial waste leakage were obtained from two states. Eight soil samples were collected from two horse arenas, a residential site, and three different roadways in Missouri. One soil sample was obtained from each of two locations in New Jersey, a herbicide manufacturing plant and a scrap metal yard. This group of soil samples exhibited a wide range of physical and chemical characteristics as shown in Table 1. This wide range of characteristics facilitated interpretation of the TCDD solubility data from the contaminated soils and identification of mechanisms controlling this solubility.

Analytical Methods

Soil pH was determined in a paste using a soil to water ratio of 1:1 (McLean, 1980). Electrical conductivity (EC) was measured using a suspension of soil and water mixed in a ratio of 1:2 (Watson, 1978). Total organic carbon in soil was determined by oxidation with $K_2Cr_2O_7$ and colorimetry (Watson, 1978). Total organic matter in soil was estimated by multiplying total organic carbon by 1.742 (Allison, 1965). Cation exchange capacity (CEC) was determined by saturating the soil with NH_4^+ followed by displacement of the NH_4^+ with K^+. Analysis of displaced NH_4^+ was conducted using an ammonium ion-specific electrode (Allen et al., 1974). The specific surface area of each soil was determined by the ethylene glycol monoethyl ether (EGME) method (Cihacek and Bremmer, 1979). Soil particle size was determined by the hydrometer method (ASTM, 1972).

Total Solvent Extractable Content

Total solvent extractable content (TSEC) of soils was determined by

TABLE 1. PHYSICAL AND CHEMICAL PROPERTIES OF SOILS

No.	Location	Site	Total[a] 2,3,7,8-TCDD Concentration	Organic Matter (mg/g)			Surface Area	CEC[d]	pH	EC	Bulk Density	Sand	Silt	Clay	Texture
				TSEC[b]	Peak Area[c]	Total									
			ng/g				m^2/g	meq/100 g		µmhos/cm	g/cm^3	--------%	--------	-------	
1	New Jersey	Industrial-2	26,316	21.5	274.0	80	22.3	12.7	7.5	65.0	1.27	81.1	17.9	1.0	Loamy Sand
2	Missouri	Shenandoah Stables (Slough Area)	2,438	7.1	14.8	60	35.7	39.0	7.3	195.2	1.30	42.3	51.6	6.1	Silty Loam
3	Missouri	Piazza Road-1	1,095	5.2	1.5	27	27.6	35.2	6.7	95.0	1.34	57.4	30.1	12.5	Sandy Loam
4	New Jersey	Industrial-1	1,083	19.4	20.9	37	38.9	17.8	6.7	98.8	--	72.1	22.8	5.0	Sandy Loam
5	Missouri	Minker-2	762	5.3	3.5	25	25.3	24.4	7.7	139.3	1.29	33.9	58.8	7.3	Silty Loam
6	Missouri	Minker-1	608	7.0	3.7	22	25.6	19.5	8.0	119.2	--	55.9	37.3	6.7	Sandy Loam
7	Missouri	Bubbling Springs Stable	275	3.4	2.2	15	15.0	12.8	7.6	101.5	1.48	61.0	36.7	2.3	Sandy Loam
8	Missouri	Piazza Road-2	234	0.7	0.6	76	21.5	53.8	7.5	153.1	1.01	60.5	30.5	9.0	Sandy Loam
9	Missouri	Sontag Road	63	5.2	0.7	60	34.6	36.9	8.4	350.0	1.14	38.5	58.4	3.1	Silty Loam
10	Missouri	Times Beach	8	0.3	0.4	17	19.7	23.2	7.7	107.1	1.22	52.4	43.2	4.4	Sandy Loam

[a] Determined using modified EPA Region VII protocol (EPA, 1983).

[b] Total Solvent Extractable Content.

[c] Total peak area of solvent extractable components detected by GC-ECD analysis (unitless).

[d] Cation Exchange Capacity.

extracting a suspension of 30 g of soil in 30 mL of an aqueous solution containing 30% NaCl and 2% KH_2PO_4 with 30 mL of methyl tert-butyl ether (MTBE) for 18 hours in a tumbling apparatus (Warner, 1978). The MTBE phase was separated from the aqueous phase, dried with 2 g of anhydrous $MgSO_4$, and filtered through a 0.5 µM PTFE membrane filter. A 100 µL aliquot of the filtered extract was transferred to a tared aluminum weighing dish, evaporated to dryness under a heat lamp, and reweighed using a microbalance. This weight is reported as TSEC in Table 1. The method is suitable for extraction of most phenols, anilines, and neutral semivolatile organic compounds.

Electron Capture Response of Solvent Extract

The MTBE extract of soil was analyzed by gas chromatography (GC) using a fused silica capillary column and a ^{63}Ni electron capture detector (ECD).

The combined GC-ECD peak area of all chromatographic peaks eluting after 5 minutes, listed as Peak Area in Table 1, was obtained using a computerized integration and data processing system. Since the electron capture detector detects primarily halogenated compounds, the combined peak area is an indicator of total amount of halogenated organic compounds present.

Background (uncontaminated) soil samples were collected near the locations of each of the contaminated soils. TSEC was determined for each of these background samples.

Soil Extraction Methods

The analytical method for TCDD in a soil leachate required a sample volume of 1 liter. Two batch methods and a method for leaching intact cores were used to generate soil leachate (aqueous extract) to accommodate this requirement.

The Solid Waste Extraction Procedure-Release (SWLP-R) was used to determine the effect of sequential extraction on the solubility of TCDD from soil (Garrett, et al., 1984). This method is similar to the Standard Leaching Test (SLT) Procedure R developed by Ham et al. (1979), wherein the same soil sample is re-extracted with fresh water. For this procedure, a 100 g aliquot of soil was placed in a 2.7-L glass bottle with 1 L of distilled water. The bottle, with the soil-water suspension was continuously tumbled for 18 hours using an electric-powered rotary extractor set at 30 rpm. The suspensions were pressure-filtered through hydrophilic cellulose acetate and nitrate membranes with a pore size of 0.45 µM. Each soil sample was extracted a second time by repeating the above procedure using the same soil and fresh water. This procedure results in filtered soil leachate (aqueous extract) representing two sequential extractions from the same soil sample. Two samples of soil from each location were extracted by this procedure (SWLP-R). The soil leachates were analyzed for TCDD using a modified version of the U.S. EPA Region VII protocol (U.S. EPA, 1983).

A second version of the SWLP procedure was used, which was similar to the SLT-Procedure C (Ham, et al., 1979), wherein three fresh soil samples are extracted sequentially with the same water. The soil leachate is filtered after extracting the soil sample and then used to extract the next soil sample. This procedure is designated SWLP-C where C denotes concentration; this is done to determine the effect of a lower liquid to solid ratio on TCDD solubility in soil. Two liters of distilled water were tumbled for 18 hours with 200 g of contaminated soil. The soil leachate was pressure-filtered through a 0.45 µM membrane, and again tumbled for 18 hours with 200 g of fresh soil, followed by filtration. This procedure was repeated with a third fresh 200-g portion of soil. The final volume of soil leachate from the third filtration, because of retention by the three soil samples, ranged from 1430 mL to 1640 mL depending on the texture of the soil. The overall liquid to solid ratio for this method was approximately 2.5:1. Two sets of three soil samples from each location were extracted by this procedure (SWLP-C). The final soil leachate was analyzed for TCDD by the modified U.S. EPA Region VII protocol.

Intact soil cores were leached with distilled water for comparison with results from the two SWLP procedures and to study the effect of soil structure and secondary porosity on soil-liquid contact and TCDD leaching. One liter of distilled water was allowed to percolate through the soil cores. The soil leachate was filtered through a 0.45 μM membrane. A second 1-L aliquot of distilled water was percolated through each soil core by repeating the above procedure. Each 1-L aliquot, representing a liquid to solid ratio of approximately 1:1, was analyzed for TCDD.

Partition Coefficients

Partition coefficients (Kp's) measure the relative distribution of a substance between the soil solid phase and the associated aqueous phase. These coefficients are a key parameter in the predictive modeling of TCDD movement in water percolating through soil. Large partition coefficients for compounds indicate high partitioning onto the solid phase and consequently very slow rates of movement in soil.

Two partition coefficients (K_p and K_{oc}), defined below, were calculated for the contaminated soils from results of each of the batch methods and the intact core studies.

$$K_p = \frac{(\text{ng TCDD per g of soil} \times 10^3)}{(\text{ng TCDD per L of extract})} \quad (1)$$

$$K_{oc} = \frac{K_p}{(\text{g organic carbon per g of soil})} \quad (2)$$

When calculated in this manner, K_p corresponds to the constant in the Freundlich equation with the exponent equal to 1.0 (linear isotherm).

In previous studies a considerable part of the variability in K_p between different soils was strongly correlated with soil organic carbon content (Hassett et al., 1983). As a result of this strong correlation, partition coefficients are often normalized on the basis of organic carbon content (equation 2). Normalized partition coefficients (K_{oc}) were shown to have a strong linear relationship with the octanol-water partition coefficients (K_{ow}) using several nonpolar organic compounds (Hassett et al., 1983; Karickhoff et al., 1979). This relationship supports the theory that hydrophobic sorption on soil organic matter is a major process controlling retention and movement of compounds such as TCDD.

Solute Transport Model

Sufficient data on rainfall, evapotranspiration, and soil hydraulic properties were not available to make site-specific predictions of TCDD movement rates in water percolating through soil. However, using average values of hydraulic conductivity, density, and porosity from the soil cores and making assumptions to supply missing data, predictions of TCDD movement rates were made to illustrate the general magnitude of these rates and the factors that most influence them.

An analytical solution of a solute transport equation was used to predict TCDD movement rates. The governing equation for one-dimensional convective-dispersive solute transport with no production or decay is:

$$R(\partial C/\partial t) = D(\partial^2 C/\partial X^2) - v(\partial C/\partial X) \quad (3)$$

where: R = the retardation factor $(1+(\rho_b/n)K_p)$
ρ_b is the bulk density (g/cm^3)
n is the porosity (cm^3/cm^3)
K_p is the partition coefficient (mL/g)
X = the distance from the soil surface (cm)
C = concentration in solution (ng/L)
D = the dispersion coefficient (cm^2/day)
v = the pore water velocity (cm/day).
t = time (day)

The initial condition was zero solute (TCDD) concentration in the soil, i.e., C(X,0) = 0. Boundary conditions were constant flux at the soil surface and no change in concentration at infinite depth (semi-infinite system).

$$[-D(\partial C/\partial X) + vC] \quad X=0 = vC_0 \qquad (4)$$

Constant flux was selected as the boundary condition at the soil surface instead of the more common constant concentration boundary condition because van Genuchten and Alves (1982) and van Genuchten and Parker (1984) have shown that use of the constant concentration boundary condition leads to mass balance errors. Values for hydraulic conductivity (10^{-3} and 10^{-4} cm/s), bulk density (1.25 g/cm^3), and porosity (0.53 cm^3/cm^3) from the leaching behavior and physical measurements on the intact cores were used. A hydraulic gradient of 1.0 was used to represent saturated soil with no appreciable ponding of water on the surface. The dispersion coefficient was calculated from the relationship presented by Biggar and Nielsen (1976).

The retardation equation was used as a second means of predicting TCDD movement under saturated flow conditions (Freeze and Cherry, 1979). This equation is:

$$V_c = v/R \qquad (5)$$

where: V_c = contaminant velocity
v = pore water velocity
R = retardation factor.

The retardation equation is a convenient tool for estimating solute movement rates. This equation is limited to predicting the velocity of the maximum solute concentration for a slug input or the velocity of the 0.5 relative concentration (C/C_0) for a continuous input. However, relative concentrations less than 0.5 are often of interest. For example, a C/C_0 of 0.01 would correspond to TCDD solution concentrations ranging from 5.5×10^{-1} ng/L to 1×10^{-3} ng/L in leachates from soils in this study. These concentrations are substantially above suggested water quality criteria for aquatic life and human health (U.S. EPA, 1984). The analytical solution was used to allow prediction of velocities for other values of C/C_0.

Calculations for the analytical solution were performed using a Tektronix 4052A computer and a BASIC language program adapted from the FORTRAN program of van Genuchten and Alves (1982). Minimum, maximum, and mean K_p values from the average of the intact core leaching and SWLP-C methods were used in calculations for both equations.

RESULTS AND DISCUSSION

TCDD in Soil Leachate

TCDD concentrations in soil leachates from the three leaching methods are summarized in Table 2. For the batch extractions, TCDD concentrations in solution ranged from 0.1 ppt to 56 ppt (ng/L); 79% of the values were 2.0 ppt or less. For the intact cores, TCDD concentrations in solution ranged from 0.1 ppt to 14 ppt; 81% of the values were 2.0 ppt or less.

The mean values of 1.0 and 1.3 ng/L for the SWLP-C and intact core procedures, respectively, (Table 2) are within one standard deviation of the mean value for the first extract of SWLP-R. The former two methods use a liquid to solid ratio of less than 2.5:1. Both the intact core procedure and SWLP-C are relatively complex and time-consuming. Procedure SWLP-R with one extraction often gives equivalent results with far less effort.

Partition Coefficients

K_p and log (K_p) values computed for each soil and extraction procedure are listed in Table 3. Mean K_p and mean log (K_p) values for the first extract of the SWLP-R, the SWLP-C, and the Intact Core leaching procedure are very similar. The partition coefficients for the second extract of the SWLP-R are lower than values obtained by the other procedures due to higher TCDD concentrations in soil leachate. These results indicate that the first extract of the SWLP-R, the SWLP-C, and the Intact Core procedures are essentially equivalent in their representation of the partitioning of TCDD in these soils.

TABLE 2. 2,3,7,8-TCDD CONCENTRATIONS IN SOIL LEACHATES FROM SWLP-R, SWLP-C, AND THE INTACT CORE PROCEDURES

Number	SWLP-R Extract 1[a]	SWLP-R Extract 2[a]	SWLP-R Mean	SWLP-C Mean	Intact Core Extract 1[b]	Intact Core Extract 2[b]	Intact Core Mean
	ng/L						
1	5.8 (0.75)	55.3 (0.03)	30.5	3.1 (0.61)	2.0 (0.23)	2.3 (0.27)	2.1
2	9.0 (0.07)	15.1 (0.10)	12.1	1.8 (0.15)	5.8 (1.18)	4.4 (0.98)	5.1
3	0.4[c] (0.63)	0.3 (0.29)	0.3	0.5 (0.61)	0.8[c] (1.25)	0.4[c] (0.39)	0.7
4	0.9 (0.28)	1.2 (0.24)	1.0	0.2 (0.02)	ND[d]	ND	ND
5	0.7 (0.46)	1.5 (0.18)	1.1	0.9 (0.09)	ND	ND	ND
6	0.2 (0.09)	3.7 (0.16)	2.0	ND	1.8 (0.56)	1.8[c] (0.32)	1.8
7	0.5[c] (0.01)	2.1 (0.12)	1.3	0.9 (0.13)	0.2[c] (0.25)	0.4[c] (0.97)	0.3
8	0.1 (0.21)	0.1[c] (0.06)	0.1	0.1 (0.25)	0.1[c] (0.71)	0.1[c] (0.52)	0.1
9	0.2 (0.02)	0.1[c] (0.18)	0.2	0.9 (0.07)	0.6 (0.61)	0.4[c] (0.93)	0.5
10	0.2[c] (1.08)	0.2 (0.75)	0.2	0.2 (0.41)	0.1[c] (0.48)	0.1[c] (0.44)	0.1
Mean	1.8	8.0	4.9	1.0	1.4	1.2	1.3

[a] Values reported are the means of 2 measurements. Values in parentheses are the Relative Standard deviations of the measurements.

[b] Values reported are the means of 3 measurements. Values in parentheses are the Relative Standard deviations of the 3 measurements.

[c] Mean includes one or more values not meeting the detection limit criteria.

[d] Not determined.

TABLE 3. SOIL PARTITION COEFFICIENTS (K_p) FOR SOIL LEACHATES FROM THE SWLP-R, SWLP-C, AND THE INTACT CORE PROCEDURES

Site Number	SWLP-R						SWLP		Intact Core[a]	
	Extract									
	1		2							
	K_p	Log (K_p)	K_p	Log (K_p)	Mean K_p	Mean[b] Log (K_p)	K_p	Log (K_p)	K_p	Log (K_p)
	10^{-5}mL/g		10^{-5} mL/g		10^{-5} mL/g		10^{-5} mL/g		10^{-5} mL/g	
1	45.8	6.66	4.8	5.68	8.6	5.94	83.9	6.92	123.9	7.09
2	2.7	5.43	1.6	5.20	2.0	5.31	13.3	6.12	4.8	5.68
3	30.9	6.49	43.3	6.64	36.1	6.56	20.6	6.31	16.9	6.23
4	12.2	6.09	9.1	5.96	10.4	6.02	47.1	6.67	--	--
5	11.0	6.04	5.1	5.71	6.9	5.84	8.2	5.92	--	--
6	27.0	6.43	1.6	5.20	3.1	5.49	--	--	53.8	5.58
7	5.9	5.77	1.3	5.11	2.2	5.34	3.0	5.47	9.4	5.97
8	23.2	6.37	20.9	6.32	22.0	6.34	25.7	6.41	20.3	6.31
9	2.6	5.41	7.3	5.86	3.8	5.58	0.7	4.85	1.3	5.11
10	0.4	4.40	0.4	4.60	0.4	4.59	0.3	4.53	1.1	5.05
Mean	16.2	5.91	9.5	5.63	9.6	5.70[b]	22.5	5.91	22.7	5.88
Std. Dev.		0.68		0.61		0.57		0.81		0.68

[a] Mean of two consecutive leachate fractions.

[b] Means are calculated from the average leachate concentrations and not the arithmetic means of the K_p values.

Values of log (K_{OC}) were computed for each soil and compared to a predicted value provided by Dragun (1984). This predicted value was calculated using relationships between aqueous solubility, K_{OC}, and an octanol-water partition coefficient (K_{OW}). The mean log (K_{OC}) values for the SWLP-R, SWLP-C, and the Intact Core procedure were 7.39, 7.58, and 7.55; these were within one standard deviation of the value of 6.95 calculated by Dragun.

The partition coefficients in Table 3 are based on data from studies where clean water is brought into contact with contaminated soil and some of the TCDD dissolves/partitions into the water. Partition coefficients in clean soils may be different. In the field, when water moves out of contaminated soil and into clean soil, the partitioning process reverses and some of the TCDD in the water will adsorb/partition onto the clean soil while some remains in solution. As discussed later in this manuscript, we believe the data indicate that other organic compounds present in contaminated soils have a strong influence on the partitioning of TCDD between soil and water. Thus, as far as partitioning is concerned, the clean soil may be a different system than the contaminated soil. Because partition coefficients calculated in this study with desorption data from contaminated soils were within the range of values predicted from physical/chemical properties of TCDD, it is reasonable, in the absence of better data, to assume that partition coefficients calculated with data from adsorption of TCDD from extracts of these contaminated soils onto clean soil will not be significantly different than the values reported here. At other locations, where contaminated soils contain greater amounts of organic co-contaminants, there is a greater chance that the partition coefficients for clean soils may be different than for the contaminated soils.

Statistical Analyses

A correlation matrix presented in Table 4 was constructed to investigate the relationships between selected soil properties, partition coefficients, and TCDD concentrations in soil and leachates. The correlation matrix is based only on data from the Missouri soils. The two New Jersey soils were excluded from this analysis because the TSEC and Peak Area were significantly greater than for any of the Missouri soils (see Table 1); inclusion in the analysis would have biased the results by providing a correlation between two sets of data rather than within a single data set. Additionally, the sources of TCDD, organic co-contaminants, and method of contamination were different in New Jersey and in Missouri. Data from the New Jersey soils generally fit the relationships shown in Figure 1. When additional data are available from soils having TSEC and Peak Area values between those for the Missouri and New Jersey soils, the entire data set can then be reanalyzed.

Observed significance levels (OSLs) which indicate the significance of the reported correlation coefficients are provided in parentheses in Table 4. For example, an OSL value of 0.05 indicates that the corresponding correlation coefficient is significantly different from zero at a confidence level of 95% and an OSL value of 0.00 indicates a confidence level of 99.9+%.

The log soil concentration of TCDD is positively correlated with the leachate concentrations for all three extraction procedures. However, the leachate concentrations were more highly correlated with the log (Peak Area) measurement. The relationships between log (Peak Area) and the soil and leachate TCDD concentrations are illustrated in Figure 1. The least squares regression line has been superimposed on each plot. While log (Peak Area) is not well defined as a soil property, it nonetheless appears to be a most important determinant for TCDD solubility in the Missouri soils.

The relationship between soil organic matter content and the K_p values for the three leaching procedures was investigated to determine the significance of computing K_{OC} values for the soils. Correlation coefficients for organic carbon vs. K_p fromthe three leaching procedures were insignificant (SWLP-R = 0.36; SWLP-C = 0.35; intact

TABLE 4. CORRELATION MATRIX FOR SOIL PROPERTY VARIABLES VERSUS BULK SOIL AND LEACHATE VARIABLES WITH OBSERVED SIGNIFICANCE LEVELS IN PARENTHESES

Dependent Variable	Log [2,3,7,8-TCDD]: Bulk Soil	Log (Peak Area)	Log (TSEC)	Percent Organic Carbon	Percent Clay
Log [2,3,7,8-TCDD]: Bulk Soil		0.84 (0.01)	0.76 (0.03)	0.14 (0.74)	0.44 (0.30)
Log [2,3,7,8-TCDD]: SWLP-R	0.66 (0.08)	0.95 (0.00)	0.61 (0.11)	-0.16 (0.71)	-0.08 (0.80)
Log [2,3,7,8-TCDD]: Intact Core	0.80 (0.03)	0.92 (0.00)	0.87 (0.01	0.11 (0.81)	0.15 (0.78)
Log [2,3,7,8-TCDD]: SWLP-C	0.49 (0.27)	0.77 (0.04)	0.84 (0.02)	-0.21 (0.65)	-0.32 (0.45)

core = 0.17). Thus, computing K_{oc} values does not account for a significant amount of the variability associated with corresponding K_p values. However, log K_{oc} values were calculated for comparison with the value predicted from solubility data and the relation between solubility and K_{oc}.

Based on published information about soil partitioning of compounds similar to TCDD in clean soils, it was expected that clay content and total organic carbon content of soils would correlate strongly with the fraction of total TCDD solubilized by the water extract and that TCDD would be less soluble in soils containing greater amounts of clay and total organic carbon. However, there was not a clear relationship between either of these parameters and TCDD solubility from the contaminated soils used in this study. Instead, both the amount of TCDD in a soil and amount of TCDD released in a water extract were strongly related to the total solvent-extractable organic content (TSEC) in soil and to the halogenated semi-volatiles (peak area) in this solvent-extractable material.

In soils with higher TSEC and peak area, the TCDD concentrations in water extracts were greater. This occurred in spite of the fact that TSEC represented only a portion of the total soil organic matter. In the contaminated soil, TSEC ranged from 0.3 mg/g to 21.5 mg/g; total organic matter in the same soils ranged from 15 mg/g to 80 mg/g. Because TSEC for background (uncontaminated) soils was so low (0.51 to 1.54 mg/g), it is concluded that both TSEC and peak area measure materials that were in the wastes along with TCDD. This suggests that other organic contaminants in wastes have a major role in controlling the solubility and movement of TCDD in contaminated soils and that TCDD behaves differently in contaminated soils than would be predicted from studies of clean soils spiked only with TCDD.

Solute Transport Modeling

The retardation equation and the analytical solution predict the movement of TCDD from a layer of contaminated soil into an underlying layer of uncontaminated soil. The analytical solution assumes that TCDD does not reach the lower boundary of the uncontaminated soil during the time period being modeled (semi-infinite system). All depths are measured downward from the boundary between contaminated and uncontaminated soil.

The two models were used first to predict, for a single depth and a single hydraulic conductivity (10^{-4} cm/sec), the time for relative concentrations (C/C_o) to reach specified values. C_o is the concentration of TCDD in water leaving the contaminated soil and C is the concentration at 30 cm depth in the uncontaminated soil at a particular time. For example, $C/C_o = 0.5$ would mean 1.55 ng/L if the SWLP-C data for site No. 1 was being used (3.1 ng/L, Table 2). Longer times are required for C/C_o to reach the higher values (Table 5) because dispersion at the contaminant front advances the lower relative concentrations more rapidly. Differences in movement rates between various C/C_o are relatively minor compared to the direct linear effect of changes in K_p. For example, when K_p increases from 0.3×10^5 to 133.6×10^5 (a relative increase of 445) the predicted times increase by factors ranging from 444 to 447.

Next, the models were used to predict changes in concentration with depth for fixed times and a single C_o. Figure 2 shows some of these results using hydraulic conductivities of 10^{-3} and 10^{-4} cm/sec, 1.2 ng/L of TCDD in the water (C_o) coming from the contaminated soil, and the mean K_p for all soils. Even for the higher hydraulic conductivity, thousands of years are required for TCDD to move to appreciable depths.

These predictions represent worst-case conditions - continuous saturated water flow instead of the intermittent flows (both in time and direction) observed in field soils. Even under these conditions the movement rates are so slow that TCDD is essentially immobile in water percolating through soil. Other transport mechanisms such as wind and water erosion of contaminated soils are likely to be significantly more important for these soils.

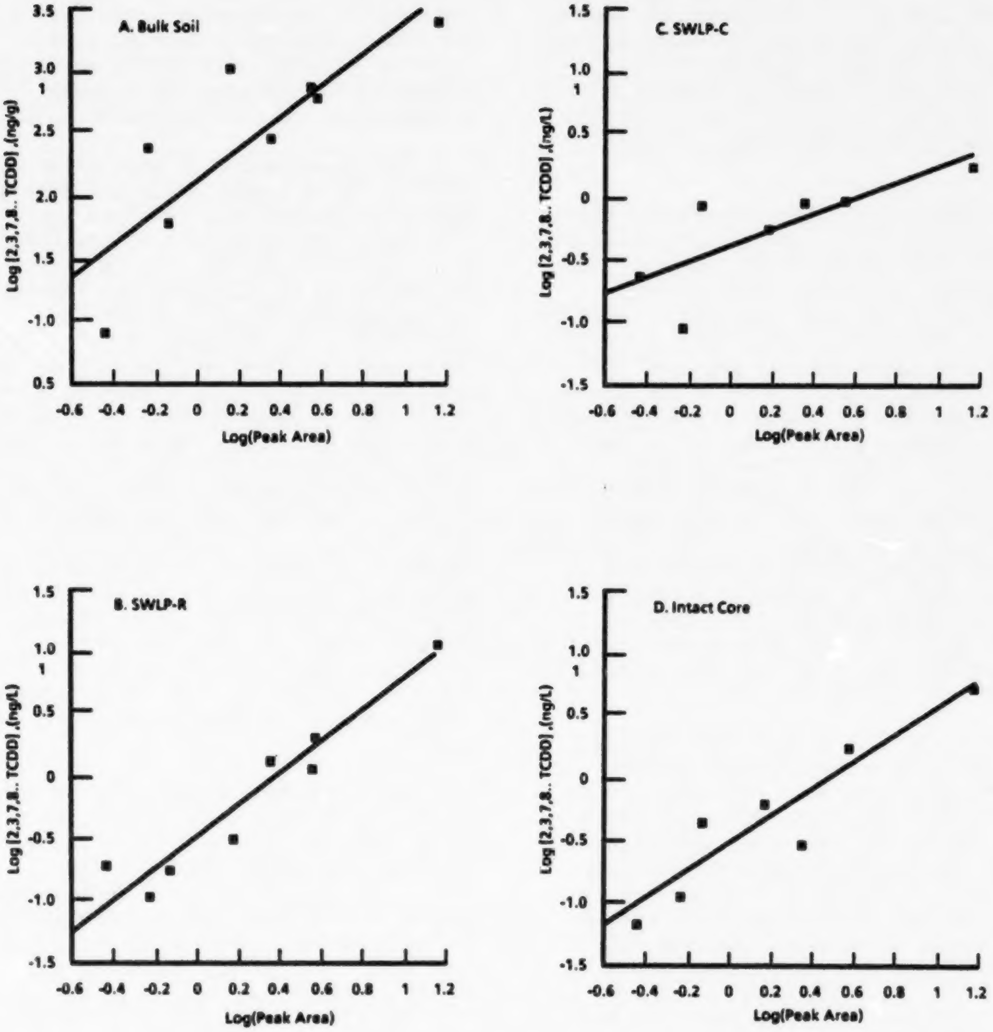

FIGURE 1. LOG-LOG PLOTS OF 2,3,7,8-TCDD CONCENTRATIONS IN (A) BULK SOIL, (B) SWLP-R, (C) SWLP-C, AND (D) INTACT CORE LEACHATE VERSUS PEAK AREA

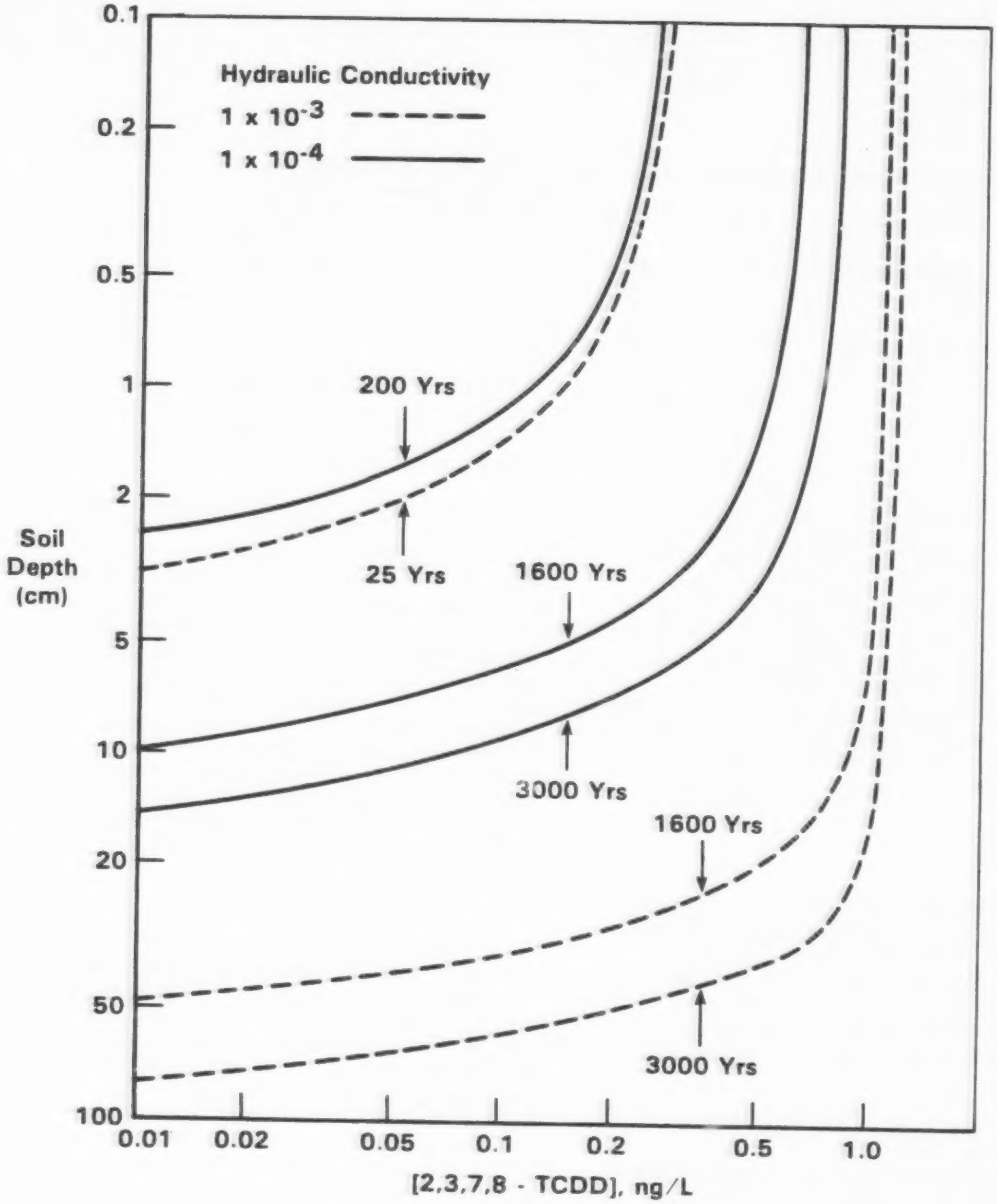

FIGURE 2. PREDICTED TRANSPORT OF 2,3,7,8-TCDD IN SOIL WHERE $K_p = 22.6 \times 10^5$ mL/g AND $C_o = 1.2$ ng/L.

TABLE 5. TIME (YRS) FOR 2,3,7,8-TCDD RELATIVE CONCENTRATION (C/C_o) TO REACH THE INDICATED VALUE AT 30 CM SOIL DEPTH.

Equation	C/C_o	Kp x 10^{-5}, mL/g		
		133	22.6	0.3
Retardation	0.5	1.59×10^5	2.69×10^4	3.57×10^2
Analytical	0.5	1.61×10^5	2.72×10^4	3.61×10^2
	0.1	8.66×10^4	1.46×10^4	1.95×10^2
	0.01	5.39×10^4	9.05×10^3	1.21×10^2

As discussed above, K_p for TCDD in contaminated water moving in clean soil may be different than the K_p used in these predictions. However, it is unlikely that they will be significantly less or that movement rates will be significantly greater unless there are large amounts of other organic contaminants present in the soil with the TCDD and the TSEC and Peak Area are much greater than the values measured for these soils.

CONCLUSIONS

Soil properties such as clay content and organic matter content were not significantly correlated with TCDD soil partition coefficients. Previous investigations using clean soils spiked with TCDD and similar nonpolar compounds indicated that a strong correlation should be expected.

A strong association was found between TCDD concentration in soil leachates (aqueous extracts) and solvent-extractable halogenated semivolatile organic matter content of soils as measured by TSEC and Peak Area. Because the TSEC and Peak Area of background soils were much lower than for the contaminated soils, it is concluded that these parameters represent organic materials that were applied to soil in wastes along with the TCDD. The strong association between soluble TCDD and solvent-extractable material suggests that TCDD dissolution from contaminated soils is regulated by these halogenated semivolatiles.

The three extraction methods used for determining TCDD partitioning coefficients provided essentially identical results when only the first extract from the SWLP-R procedure was compared with results from the SWLP-C and the Intact Core procedure. The second extract in the SWLP-R procedure resulted in greater TCDD solution concentrations in most cases. These increased TCDD concentrations were ascribed to the effect of soil electrolytes on the activity coefficients and solubility. Based on these results, a single extract using the SWLP-R procedure is the most cost-effective means for determining TCDD partitioning . coefficients.

When the TCDD partition coefficients are used with two solute transport models, the predicted worst-case TCDD movement rates in water percolating through soil are so slow that other transport mechanisms such as wind and water erosion are likely to be more significant. Partition coefficients and TCDD movement rates may be different in soils that have greater amounts of halogenated semivolatile organic compounds than the soils used in this study.

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DEVELOPMENT OF STANDARDIZED BATCH ADSORPTION PROCEDURES: EXPERIMENTAL CONSIDERATIONS

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ABSTRACT

A Technical Resource Document (TRD) is being developed by the authors to provide information on the use of batch procedures for determining the adsorption affinity of soils for solutes in waste leachates. The TRD will be part of the technical basis for guidance by the Office of Solid Waste on design and evaluation of compacted soil liners for waste storage and disposal facilities. In developing the batch procedures for the TRD it was found that the method of mixing the solute-soil mixtures influenced their adsorption by soil; more vigorous shaking resulted in greater adsorption. The National Bureau of Standards rotating tumbler was adopted as the method of choice. A mixing interval of 24 hours appears to be a valid operational definition of equilibrium for these types of batch procedures. Changes in solution concentration of arsenic and cadmium exposed to different soil materials were negligible after 10 hours. The selection of appropriate soil:solution ratios for inorganic solutes must be found empirically, but suitable ratios for hydrophobic organic solutes can be estimated from the compounds' solubility in water and the organic carbon content of the soil.

INTRODUCTION

This paper discusses the effect of method of mixing, equilibration time, and soil:solution ratio on results from batch studies of the adsorption capacity of soils for solutes in waste leachates. This work is part of the development of a Technical Resource Document (TRD) on use of batch procedures in designing compacted soil liners for removal of solutes from waste leachates. The TRD will be part of the technical basis for guidance by the Office of Solid Waste on design and evaluation of compacted soil liners for waste storage and disposal facilities.

The capacity of geological materials to attenuate potential pollutants has been studied by many researchers, especially during the last 30 years. The batch adsorption or batch equilibrium technique has often been used in laboratory studies to assess the capacity of soils and soil components to attenuate chemical constituents in solution. However, the batch procedures used in these studies vary considerably in terms of experimental conditions, and in some cases may yield different results even when the same soils, solutes, and concentrations are studied.

In principle, the batch adsorption technique is relatively simple, accounting, in part, for its popularity. This technique consists of mixing a solution containing aqueous solutes of known composition and concentration with a given mass of adsorbent for a period of time. The solution is then separated from the adsorbent and chemically analyzed to determine changes in solute concentration. However, a number of experimental parameters may affect the adsorption of a given constituent. For inorganic solutes, these parameters include time, temperature, method of mixing, soil:solution ratio, moisture content, and the composition and concentration of other dissolved constituents in the solution (White, 1966; Barrow and Shaw, 1975, 1979;

Helyar et al., 1976; Hope and Syers, 1976; Griffin and Au, 1977; Barrow, 1978; Ainsworth et al., 1984, and Roy et al., 1984). For organic solutes, similar parameters may also affect the adsorption (Bailey and White, 1970; Grover and Hance, 1970; Dao and Lavy, 1978; Koskinen and Cheng, 1983; and Horzempa and DiToro, 1983). In addition, solution pH, adsorbate volatility, photodegradation, biodegradation, and stability can also affect the adsorption of organic solutes (Harris and Warren, 1964; Chou and Griffin, 1983; and Scott et al., 1982).

Soil:solution ratios used in batch procedures have varied from 1:10,000 for the adsorption of phosphates by gibbsite (Muljadi et al., 1966) to 1:1 for the adsorption of nickel by a soil studied by Bowman et al. (1981). Equilibration time, another basic experimental parameter in batch techniques, has varied from 30 minutes, for the adsorption of zinc by soil investigated by Shukla and Mittal (1979), to 6 days to equilibrate a phosphate-soil system investigated by Jones et al., (1979). These experimental conditions were probably appropriate for the particular system under study, but the differences in conditions make comparisons of data among studies difficult.

It is apparent from the literature that there is currently no standard batch method for the collection of adsorption data. The goal of this project is to fill this gap by developing relatively simple batch adsorption procedures and guidelines for their use. These products are intended for use by the EPA's Office of Solid Waste in developing guidance on design and evaluation of compacted soil liners for waste storage and disposal facilities. The results of this study will be included in a Technical Resource Document describing the use of batch procedures, scientific basis and rationale for the procedures, and interpretating adsorption data generated by each procedure.

CURRENT STATUS

Cooperative work with ASTM Committee D34 (Waste Disposal) is being conducted at a number of laboratories to develop and test a batch procedure for determining the affinity of soils for solutes. Round-robin trials were conducted under the auspices of ASTM subcommittee D34.02.03 (Predictive Methods). On the basis of these trials a batch procedure has evolved for measuring a 24-hour distribution ratio (R_d) for soils and sediments at one initial solute concentration and provides one point on an adsorption isotherm. This proposed ASTM procedure is being used as the basis for an expanded procedure that will generate data at several concentrations so that an adsorption isotherm can be constructed. Throughout the rest of this paper, the term "24-hour R_d" will be used to identify the single point (ASTM) batch adsorption procedure proposed for use in the TRD.

The 24-hour R_d procedure consists of agitating a soil:solution mixture of 1:20 by using a National Bureau of Standards (NBS) rotating tumbler for 24 hours. The air-dried sample is initially passed through a 2-mm sieve. During the mixing interval, neither the pH nor the ionic strength of the mixture is controlled. The tumbler is operated at room temperature (22° ± 5°C). After 24 hours, the mixture is either filtered or centrifuged and the resulting solution analyzed chemically.

The project is nearing completion; previous papers on the project reported findings from a comprehensive literature review and the initial laboratory studies (Ainsworth et al., 1984). A paper by Roy et al. (1984) discussed how competitive interactions between solutes can influence adsorption data. The information from these publications will be integrated into the TRD.

The present paper discusses the effects of mixing method (shaker vs NBS tumbler), equlibration time, and soil:solution ratio on results from batch studies of the adsorption capacity of soils.

Mixing Method

A first generation procedure was formulated around the ASTM-A, Water Shake extraction procedure (ASTM, 1979). A sensitivity analysis of this early procedure made by a number of laboratories found that the method of mixing influenced the amount of cadmium adsorbed by the Catlin silt loam; when the shaking was more vigorous greater amounts of cadmium were adsorbed (Ainsworth et al., 1984). To improve the consistency of interlaboratory results, a National Bureau of Standards (NBS) rotating tumbler was tested as the mixing system, and a second sensitivity analysis was carried out using NBS tumblers (Table 1) and the Catlin soil. The interlaboratory results were in good

TABLE 1. Cadmium adsorption by the Catlin silt loam where a NBS tumbler was used by each laboratory.

Lab	Initial conc.	24-hr conc.	Rd	Lab	Initial conc.	24-hr conc	Rd
	--------mg/L------		mL/g		--------mg/L-------		mL/g
A.				A.			
Trial 1	200	35.7	92.0	1	10.1	0.114	1734
2	200	36.2	90.5	2	10.1	0.126	1567
3	200	34.6	95.6	3	10.1	0.125	1580
	mean	35.5	92.7		mean	0.122	1627
	std. dev.	±0.8	±2.6		std. dev.	±0.01	±92.8
B.				B.			
Trial 1	200	31.8	105.8	1	10.0	0.110	1798
2	200	35.8	91.7	2	10.0	0.135	1461
3	200	36.8	88.7	3	10.0	0.165	1214
	mean	34.8	95.4		mean	0.136	1491
	std. dev.	±2.6	±9.1		std. dev.	±0.026	±293
C.				C.			
Trial 1	200	35.6	92.4	1	10.0	0.127	1554
2	200	35.6	92.4	2	10.0	0.127	1554
3	200	35.0	94.3	3	10.0	0.132	1495
	mean	35.4	93.0		mean	0.128	1534
	std. dev.	±0.4	±1.1		std. dev.	±0.003	±34.0
D.				D.			
Trial 1	190	31.0	102.5	1	9.8	0.130	1487
2	190	30.0	106.6	2	9.8	0.110	1761
3	190	31.0	102.5	3	9.8	0.120	1613
	mean	30.6	103.9		mean	0.120	1620
	std. dev.	±0.6	±2.4		std. dev.	±0.01	±137
	overall mean	34.1	96.3		overall mean	0.126	1568
	std. dev.	±2.3	±5.2		std. dev.	±0.01	±66.5
	% CV	6.9	5.4		% CV	7.94	4.2

agreement. The coefficient of variation (% CV) between mean values for each laboratory reflects, in part, the precision in the mixing method. The coefficient of variation of R_d values, based on initial cadmium concentrations of 10 mg/L and 200 mg/L, were less than 10%. The interlaboratory coefficient of variation for similar concentrations, using the shaking method, was more than 125% in interlaboratory comparisons (Table 2). Because all other parts of the procedure

TABLE 2. Cadmium adsorption by the Catlin silt loam where shakers were used by each laboratory.

Lab		Initial conc.	24-hr conc.	Shaker rate	
		---------mg/L--------		strokes/min	throw (cm)
A.				59	7.62
Trial 1		200	16.9		
2		200	13.2		
3		200	11.4		
	mean		13.8		
	std. dev.		±2.8		
B.				70	3.81
Trial 1		200	83.8		
2		200	88.2		
3		200	86.7		
	mean		86.2		
	std. dev.		±2.2		
C.				100	3.18
Trial 1		200	1.88		
2		200	1.77		
3		200	1.70		
	mean		1.78		
	std. dev.		±0.09		
D.				70	3.18
Trial 1		200	26.5		
2		200	21.5		
3		200	10.0		
	mean		19.3		
	std. dev.		±8.46		
	overall mean		30.3		
	std. dev.		38.0		
	% CV		125.5		

were the same in both cases, the mixing method was the primary contributor to variation in the interlaboratory means. The NBS tumbler was adopted as the method of choice because of the much lower coefficient of variation between laboratory means.

Equilibration Time

The proposed 24-hour R_d procedure provides a measurement of adsorption after 24 hours. This may or may not be long enough for the development of true chemical equilibrium or steady state. To evaluate the partitioning of solutes and adsorbates in systems at equilibrium, the time interval necessary for the development of equilibrium must be experimentally measured. As indicated earlier, some investigators have used equilibration times of days or even weeks. However, adsorption *per se* is generally regarded as a fast reaction, and subsequent removal of a solute from solution may be attributed to other processes. The ambiguity in the definition and measurement of equilibration times has been acknowledged as a major problem in adsorption studies (Anderson et al., 1981). The EPA (U.S. EPA, 1982) suggested that the equilibrium time should be the minimum amount of time needed to establish a rate of change of the solution concentration equal to or less than 5% per 24-hour interval. The efficacy of this operational definition for equilibrium was evaluated, using seven soil materials. Each of the adsorbents was exposed to arsenic and cadmium solutions, initially containing 200 mg/L, for periods of up to 72 hours. The solutions were analyzed, and the rate of removal of the solute was determined. All of the soil-solute systems in this study were found to be in equilibrium as defined by this operational definition. Figure 1 presents representative data for cadmium adsorption.

Soil:Solution Ratio

As previously mentioned, the soil:solution ratio of the proposed 24-hour R_d procedure is 1:20. However, a single soil:solution ratio cannot be used satisfactorily in all cases.

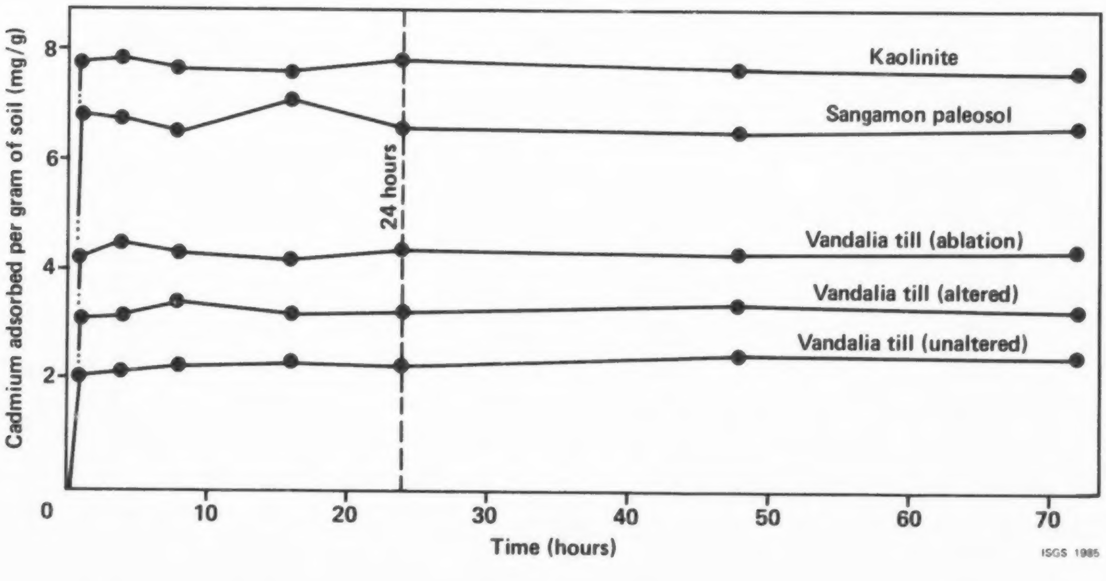

Figure 1. Equilibration time determinations for a cadmium solution of 200 mg/L contacted with various soil materials.

During the development of procedures togenerate adsorption isotherms, it became necessary to determine soil:solution ratios that would permit enough solute to be adsorbed to result in measurable, statistically significant differences in solution concentration. Unfortunately, with inorganic compounds, this cannot be determined *a priori*.

An empirical, systematic procedure was developed to determine a suitable ratio for a given soil-water system and concentration range. A value of 10% to 30% adsorption for the highest concentration used is a useful criterion. This will give a discernible decrease in solute concentration that is statistically acceptable with respect to the initial concentration. An example of this type of approach is given in Table 3. Using a 1:4 soil:solution ratio, more than 90% of the cadmium initially added (200 mg/L) was adsorbed by both the Sangamon paleosol and the Vandalia till. If the 1:4 soil:solution ratio were used to generate data at lower concentrations than the 200 mg/L used in this example, those points may be at concentrations below analytical detection limits. In contrast, when a 1:500 ratio was used at the lower concentration (10 mg/L), about 60% of the cadmium initially added was adsorbed by the Sangamon sample. However, when the 200 mg/L Cd solution was used, only 9.8% was adsorbed at the same soil:solution ratio. Essentially, the object is to select a soil:solution ratio that will serve as a compromise. In this case a ratio of 1:100 was chosen to generate an adsorption isotherm because the amount of cadmium adsorbed from the high concentration range of the isotherm (200 mg/L solution) was approximately between 10 and 30%, and at the same time the amount of cadmium remaining from a low concentration (10 mg/L) solution was

TABLE 3. Soil:solution ratio determination for the Sangamon soil and Vandalia Ablation till using Cd as the adsorbate.

Soil:solution Ratio	SANGAMON Cd adsorbed µg/g	 Cd adsorbed %	VANDALIA (ABLATION) Cd adsorbed µg/g	 Cd adsorbed %
Initial concentration - 200 mg/L				
1:4	722	95.2	635	94.1
1:10	1631	86.1	1359	76.2
1:20	2792	73.7	2143	44.3
1:40	4246	56.0	3012	25.4
1:60	5165	45.4	3441	19.1
1:100	6250	33.0	3880	13.0
1:200	7500	19.8	4560	8.0
1:500	9250	9.8	4900	4.6
Initial concentration - 10 mg/L				
1:100	957	91.1	840	80.0
1:200	1736	82.7	1474	70.2
1:500	3178	60.5	2215	42.2
1:1000	4325	41.2	--	--

also within analytical detection limits. It is recommended that different soil:solution ratios not be used to generate data points for the same adsorption isotherm. Incongruent results due to the effects of competitive interactions may be observed (see Roy et al., 1984).

Although finding a suitable soil:solution ratio for inorganic solutes must be done empirically, a soil:solution ratio for hydrophobic organic solutes may be calculated if the organic carbon content of the adsorbent (soil), and the water solubility of the solute are known (McCall, 1981). A derivation of this estimation technique begins with:

$$\text{let } K_d = \frac{\mu g_a \text{ adsorbate/g soil}}{\mu g_s \text{ solute/g solution}} = \frac{\mu g_a/g}{\mu g_s/g} \qquad [1]$$

Also, if we let R = g soil/g aqueous solution [2]

then eq. [1] becomes

$$K_d = \frac{\mu g_a}{\mu g_s} \frac{1}{R} \qquad [3]$$

Since $\mu g_a + \mu g_s$ must equal the total mass of solute initially in solution (μg°), then

$$K_d = \frac{\mu g_a}{(\mu g^0 - \mu g_a)} \frac{1}{R} \qquad [4]$$

or

$$R = \frac{\mu g_a}{(\mu g^0 - \mu g_a)\, K_d} \qquad [5]$$

Thus, it is possible to select an appropriate soil:solution ratio (R) based on an estimate of the K_d (partition coefficient) of the organic solute from preliminary studies or literature values.

Partition coefficients for volatile organic solutes may be estimated from the empirical equation of Hassett et al. (1983), relating the solubility (S) of the solute in water to its organic carbon-solution partition coefficient (K_{oc}), viz.

$$\log K_{oc} = 3.95 - 0.62 \log(S \text{ in mg/L}) \qquad [6]$$

The K_d can then be calculated if the organic carbon content (TOC) of the adsorbent is known by

$$K_d = K_{oc}\,(\%\ \text{TOC})/100 \qquad [7]$$

To evaluate the efficacy of this approach (eqs. [1-7]) soil:solution ratios for dichloroethane, tetrachloroethylene, and o-xylene were calculated, then these results were compared to experimental results obtained by using the Catlin silt loam (4.04% total organic carbon) as the adsorbent.

The solubility of dichloroethane and tetrachloroethylene is 8450 mg/L and 200 mg/L, respectively, at 25°C (Chiou et al., 1979), and the solubility of o-xylene is approximately 175 mg/L at 25°C (McAuliffe, 1966). From equations [6] and [7], the calculated K_d values of dichloroethane, tetrachloroethylene, and o-xylene were approximately 1.3, 13.4, and 14.7, respectively. In order to obtain 20% adsorption ($\mu g_a/\mu g^\circ = 0.2$), the estimated soil:solution ratios would be

dichloroethane:

$$R = \frac{20}{80} \frac{(\ 1\ )}{(1.3)} = \frac{1}{5.3}$$

tetrachloroethylene:

$$R = \frac{20}{80} \frac{(\ 1\ )}{(13.4)} = \frac{1}{53.5}$$

o-xylene:

$$R = \frac{20}{80} \frac{(\ 1\ )}{(14.7)} = \frac{1}{58.7}$$

From these results, a soil:solution ratio of 1:50 was used for a mixture containing all three compounds to generate adsorption isotherms for each organic compound (Figs. 2 and 3). It was felt that a tri-solute system would be more representative of leachate samples taken in the field. Over the concentration range studied, the adsorption of dichloroethane, tetrachloroethylene, and o-xylene by the Catlin silt loam could be described by the linear relationship

$$\frac{x}{m} = K_d C \qquad [8]$$

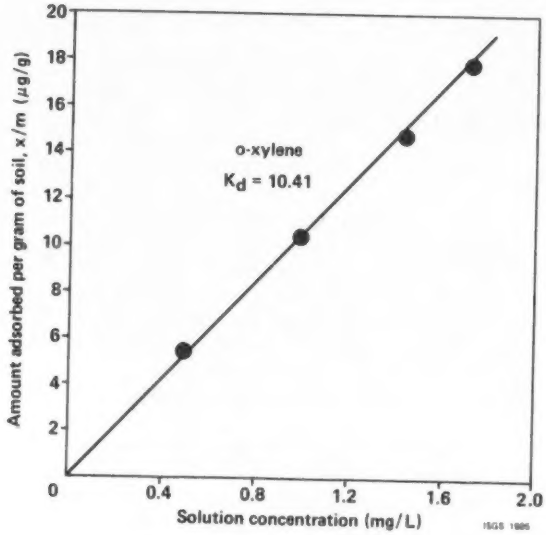

Figure 2. Adsorption isotherm of o-xylene by the Catlin silt loam at 23°C.

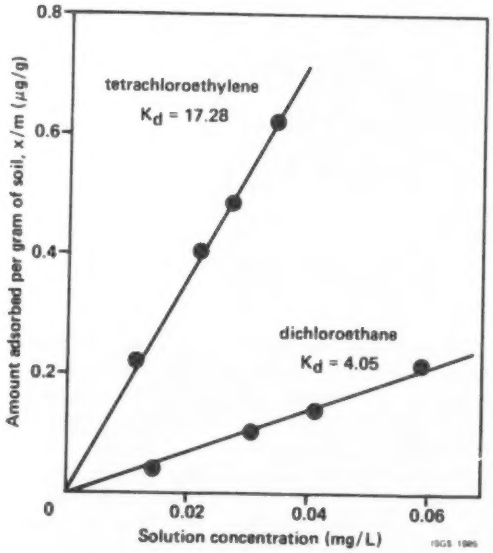

Figure 3. Adsorption isotherms of dichloroethane and tetrachloroethylene by the Catlin silt loam at 23°C.

where x/m is the amount of solute adsorbed per unit weight of adsorbent (µg/g), K_d is an adsorption constant (mL/g), and C is the equilibrium concentration (µg/mL). Equation [8] represents a special case of the Freundlich adsorption equation ($x/m = K_f C^{1/n}$) where the 1/n term is equal to one, and K_f reduces to a simple distribution coefficient (K_d) for all three compounds (Figs. 2 and 3).

Based on the experimental data, a soil:solution ratio (R) can be recalculated from the percentage of each compound adsorbed using eq. [4]:

dichloroethane:

$$R = \frac{6.55}{100-6.55}\left(\frac{1}{4.05}\right) = \frac{1}{58}$$

tetrachloroethylene:

$$R = \frac{26.59}{100-26.59}\left(\frac{1}{17.28}\right) = \frac{1}{48}$$

o-xylene:

$$R = \frac{16.27}{100-17.28}\left(\frac{1}{10.41}\right) = \frac{1}{50}$$

The differences in solute concentrations were statistically significant. The soil:solution ratio used in the experiments was very close to the ratio calculated from the aqueous solubility of the compounds and the organic carbon content of the soil. Thus, this estimation technique appears to yield suitable soil:solution ratios.

CONCLUSIONS

In this study, the method of mixing the solute-soil mixtures has been shown to affect the adsorption data generated from batch equilibration techniques. Proposed batch procedures that will be described in the TRD will use the National Bureau of Standards rotating tumbler as the method of choice for mixing the soil-solution slurry because there was less variability in the results using this method than when shaking methods are used. An equilibration interval of 24 hours appears to be a valid operational definition of chemical equilibrium for standardized batch procedures. Soil:solution ratios for inorganic solutes must be found empirically, but suitable ratios for hydrophobic organic solutes may be estimated from the compounds water solubility and the organic carbon content of the soil.

ACKNOWLEDGMENTS

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CLAY LINER SYSTEMS: CURRENT DESIGN AND CONSTRUCTION PRACTICES

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ABSTRACT

Objective

Information on clay liner design and construction for hazardous waste facilities was collected as part of the development of the Technical Resource Document: Design, Construction, Maintenance, and Evaluation of Clay Liners for Hazardous Waste Facilities. Critical aspects of construction that affect liner performance (e.g. seepage rates) were identified, in part, from the literature and, more extensively, from interviews with individuals currently involved in design and construction of natural clay liners and admixed bentonite clay liners for waste disposal facilities.

Literature Review

There is much information available on construction practices developed to achieve maximum bearing strength in cohesive (clay) soils for applications such as roadways and foundations. This information, however, has little application to design and construction of clay liners where the main objective is minimum permeability. Except for construction of dam cores, there have been few field examinations of clay construction related to minimum permeability. Several papers on the theory of compaction have information applicable to design and construction of low permeability liners. Among the critical elements in construction of low permeability clay liners are molding water content (wet of optimum for minimum permeability), compactive effort (interaction of duration and intensity), soil preparation (clod size, moisture control), method of compaction (sheeps foot roller, flat roller), post-construction effects (dessication cracking, damage by equipment), and uniformity of spreading and mixing for admixed liners (in-place, central plant).

Interviews

The results of the literature review were used to guide selection of topics for interviews. These topics were discussed in interviews with more than 30 geotechnical engineers and engineering geologists representing 15 engineering and waste management firms and several regulatory agencies. Many procedures identified in the literature review as optimum for achieving minimum permeability were not completely put into practice in the field because of operational difficulties (e.g. homogenizing large volumes of soil) or because data from field studies were not available to establish a quantitative relationship between individual aspects of construction practice and performance of a completed liner. Some potential construction problems and preventive measures are identified in Table 1.

Conclusions

- The relationships between typical/optimal construction practices and long-term performance of the completed liner are generally acknowledged but have not been verified through field studies.

- Leakage of several clay-lined facilities has been documented; the causes for the failures have not been established.

- Many existing facilities are functioning satisfactorily. The reason for performance differences between functioning and leaking facilities is not known due to a lack of post-failure analysis and detailed construction documentation.

TABLE 1. POTENTIAL CLAY LINER DESIGN AND INSTALLATION PROBLEMS AND PREVENTIVE MEASURES

PROBLEMS	PREVENTIVE MEASURES
Sidewall slump and collapse caused by improper sidewall design due to poor characterization of soil strength profile and/or high inward hydraulic pressure on sidewalls (sites below water table).	o Proper characterization of subsurface conditions. o Design a more gentle sidewall slope (depending on shear strength of foundation soil). o Reduce hydraulic head by: - Installing slurry wall around site perimeter to cut off groundwater. - Trenching and pumping around site to cut off groundwater. - Pumping from wells to lower local groundwater table.
Bottom heave or rupture from high inward pressure on bottom (sites below water table).	o Control depth of excavation to lower head potential. o Reduce hydraulic head by slurry wall, pumping, or other technique. o Fill landfill before heaving occurs.
Accumulation of water in landfill during construction due to rainfall or seepage into site (sites below water table).	o Cover site with inflatable dome (reduces leachate o treatment requirements for continuous operation facilities). o Seal-roll liner at end of construction day to assure proper runoff of rainfall into sump. o Reduce hydraulic head in surrounding soil. o For all infiltration: - Use leachate collection system to remove water. - Design leachate collection system or detection system (between liners) to handle extra water input.
Drying and cracking of clay liner greatly increasing permeability (dessication cracking).	o Do not construct during extremely hot, dry periods. o Wet liner during dry periods. o Cover liner with plastic sheet or soil layer if liner construction is interrupted. o Do not leave liner exposed prior to waste emplacement or leachate collection system installation.
Freeze/thaw cycling resulting in loss of liner density (increased permeability).	o Do not construct during winter in cold climates. o Cover liner with soil blanket or other insulation material when cold weather is anticipated.
Reduction in clay workability due to low temperature.	o Increase compactive effort o Cease construction till spring.
Wave erosion of upper liner after construction (liquid impoundments).	o Place rip-rap on lagoon sidewalls extending from below liquid level into the freeboard area.
Visible partings between improperly joined liner lifts and increased permeability parallel to lifts.	o Scarify or disc lower lift prior to installing next lift. o Insure moisture content of last lift and lift being installed are the same.
Pockets of high-permeability material (e.g. sand and gravel) in liner material.	o Closely inspect liner material at borrow site or as it is being installed and reject coarse-grained material.
Leaks due to improper sealing around penetrating objects.	o Avoid liner penetrations in design. o Seal properly around penetrating objects.

EFFECTS OF INORGANIC LEACHATES ON CLAY SOIL PERMEABILITY

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ABSTRACT

Hydraulic conductivity laboratory tests results for three clay soils exposed to two inorganic chemicals are documented, the potential usefulness of five additional laboratory tests for predicting clay/chemical compatibility is discussed, and an extensive literature review is summarized. Many researchers have studied the effects that organic solvents have on the hydraulic conductivity of compacted clays, and results indicate many of these solvents can cause increases in the hydraulic conductivities. Little information, however, is available in the literature concerning the effects inorganic chemicals have on the hydraulic conductivities of clays. In this research project, the hydraulic conductivities of three field clays; White Store, Hoytville, and Faceville permeated with ferric chloride (500 mg/l) and nickel nitrate (50 mg/l & 300,000 mg/l) were determined using both fixed and flexible-wall permeameters. Results indicate that at the concentrations tested, neither chemical significantly altered the hydraulic conductivities of the three clay soils. The predictive test results are seen to be inconclusive because, as indicated, no changes in hydraulic conductivities were observed.

INTRODUCTION

Hydraulic conductivity is an important parameter in the evaluation of clay soils as candidate materials for barriers to contain hazardous wastes deposited in landfills, pits, ponds, and lagoons. In research and commercial laboratories across the nation, this parameter is studied using fixed-wall and flexible-wall test cells. Other equipment designs and operating procedures used historically for water-clay interaction tests have been modified in several ways to study the effects of artificially contaminated aqueous fluids on clay soils.

The literature review conducted for this research project summarized reports that organic solvents can cause significant increases in the hydraulic conductivities of clays, while very little documentation is reported concerning the effects of inorganic liquids (1).

The literature review conducted for this research project focused on five areas: 1) flow of liquid through porous media, 2) laboratory equipment, 3) factors impacting hydraulic conductivity, 4) failure mechanisms potentially impacting clay liners, and 5) hydraulic conductivity laboratory test results. Numerous researchers have studied the impact of organic chemicals on clay hydraulic conductivity in the laboratory while far fewer have examined the effects of inorganic chemicals.

In the study of organic chemical/clay combinations, Brown and Anderson have investigated the effects of various organic liquids on the hydraulic conductivities of several clay soils (1). Large increases in hydraulic conductivity were observed for all soils tested when acetone, aniline, methanol, and xylene were used as permeant solutions. Daniel and co-workers are currently involved in testing three clays to determine the effects that hydraulic gradient, permeant liquid, and type of permeameter have on the hydraulic conductivity of compated clay (5). The three clays, a mixed cation illite, a non-calcareous smectitic clay, and a commercially processed kaolinite are being tested at hydraulic gradients of 10, 50, 100, and 300. The hydraulic conductivities are being determined using fixed-wall permeameters, flexible-wall permeameters, and consolidometers with water (0.01 N calcium sulfate), heptane, and methanol being used as permeants. Preliminary results show that at high hydraulic gradients (150 or larger), kaolinite has a higher hydraulic conductivity to methanol than to water regardless of permeameter type.

Most of the research on the effect inorganic chemicals have on the hydraulic conductivity of soils has been conducted by commercial laboratories or by individuals focusing only on sodium (11). Simons et al. have undertaken an ambitious testing program, but have to date only conducted tests in an apparatus with semi-fixed walls in which the sample is surrounded by paraffin and no data are presently available (12). Another group addressing this issue is active at The Danish Water Quality Institute; to date no reliable clay hydraulic conductivity data have been reported (6).

At Duke University, three previous studies have been conducted to test the effects inorganic chemicals have on the hydraulic conductivities of clays. Coia tested the effects of electroplating wastes on the hydraulic conductivities of four clays: pure montmorillonite, pure vermiculite, pure kaolinite, and White Store clay (2). An increase in hydraulic conductivity was observed for the pure montmorillonite while all the other clays exhibited a decrease. Care should be taken in interpreting Coia's data since his equipment and procedures were not well refined. Monserratte tested the effects of zinc chloride and chromic acid on the hydraulic conductivities of processed bentonite and White Store clays (8). No significant changes in hydraulic conductivity were observed. Tulis tested four clays; Wyoming Bentonite, Groleg Kaoline, vermiculite, and White Store clay, with metal hydroxides (13). Test results show an increase in hydraulic conductivity for the bentonite exposed to alkaline permeants while the kaolin's hydraulic conductivity decreased upon permeation with the basic solutions.

Dunn and Mitchell tested two silty clay soils which had been selected as potential liner construction materials for waste containment facilities (4). Tap water and a synthetic acidic lead-zinc tailings leachate were used as permeants. In all tests a slow decrease in hydraulic conductivity was observed. The hydraulic conductivity was also found to decrease somewhat with increasing gradient and the decrease was found to be irreversible with decreasing gradient.

Clays rich in kaolinite and illite were tested by Laguros and Robertson (7). Leachates consisting of 15 N sulfuric acid and 1.5 N sodium hydroxide created increases close to three orders and two orders of magnitude respectively.

Commercial laboratories reportedly have conducted a great number of tests on different clays and leachates. Most of the test results are not generally available because of confidentiality concerns, although some data are available. The permeants typically are complex mixtures of chemicals of widely varying characteristics. For example, D'Appolonia Inc. has compiled a lengthy list of test results (3). In some cases a slight change in hydraulic conductivity was observed, however no drastic changes occurred.

While the literature indicates that organic solvents can cause increases in the hydraulic conductivities of compacted clays, very little has been reported concerning the effects of inorganic liquids. Most of the information available on the effects of inorganics comes from irrigation studies focusing on the effects of sodium. While this information is helpful, care must be used in extrapolating this data to compacted clay liners for hazardous waste management facilities.

PURPOSE

The objectives of this reasearch project were to:

1. Document literature published on the impact of inorganic solutions on clay soil hydraulic conductivity.

2. Generate laboratory test data on the ability of selected clay soils to prevent the migration of identified inorganic solutions.

3. Investigate the feasibility of method(s) to predict deleterious effects of inorganic solutions on clay soil hydraulic conductivities.

APPROACH

Three field clays were selected for this project; White Store clay from Durham County, North Carolina, Hoytville clay from Woods County, Ohio, and Faceville clay from Claredon County, South Carolina. These soils are dominated by montmorillonite, illite, and kaolinite clay minerals, respectively.

The selection criteria for inorganic chemicals used in the laboratory tests were specified by the U. S. EPA Project Officer. Chemicals tested:

- represent constituents of inorganic fluids found under field conditions.
- represent concentrations of inorganic fluids found under field conditions.

Based on these criteria ferric chloride (500 mg/l) and nickel nitrate (50 mg/l) were selected. A nickel nitrate solution of approximately 300,000 mg/l was also used so that the cation exchange capacities of the soils would be exceeded. A solution of 0.01 N calcium sulfate, widely used by other researchers, was used as a standard permeant liquid to obtain baseline values.

No widely accepted standards exist for measuring the hydraulic conductivity of compacted clays thus the equipment and procedures developed for this project incorporate the most successful aspects of past and present experimental techniques referenced in the literature and/or devised at Duke University (1). The equipment used are modifications to standard fixed-wall permeameters and triaxial cells. Sketches of the equipment used are shown in Figure 1.

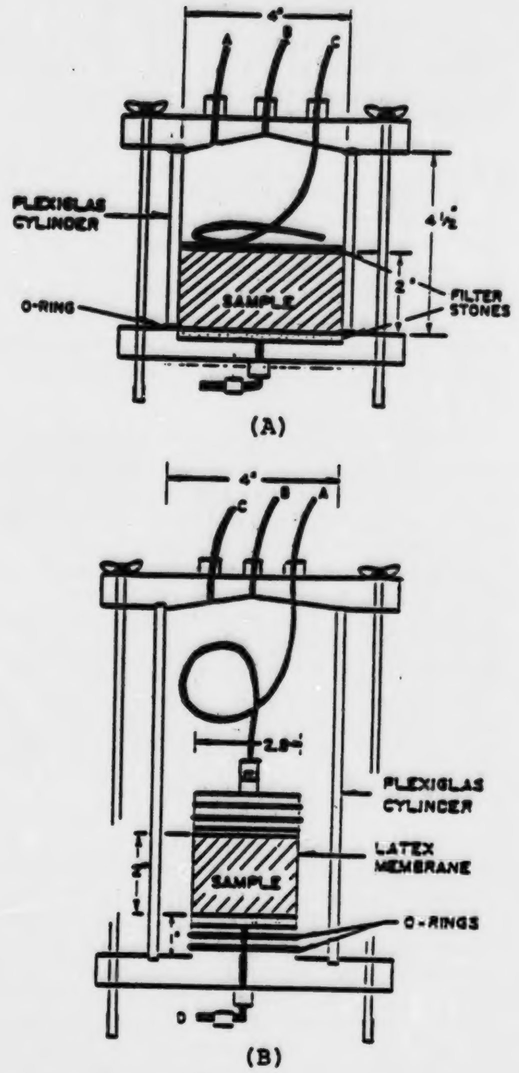

Figure 1. Fixed (A) and Flexible-wall (B) Permeameters used for Hydraulic Conductivity Testing.

These units have been designed and manufactured to meet special requirements to resist inorganic chemical attack. The hydraulic conductivity testing protocol used for this project is shown in Table 1.

TABLE 1. TEST PROTOCOL USED FOR FIXED AND FLEXIBLE-WALL HYDRAULIC CONDUCTIVITY TESTS, NUMBER OF REPLICATES

Chemical (Concentration	Fixed-Wall						Flexible-Wall					
	SHORT TERM (4-12 Weeks)			LONG TERM (12-36 Weeks)			SHORT TERM (4-12 Weeks)			LONG TERM (12-36 Weeks)		
	WS	F	H	WS	F	H	WS	F	H	WS	F	H
$CaSO_4$ (.01N)	16	14	12	2	4	2	2	2	3	1	0	0
$FeCl_3$ (500 mg/1)	6	4	4	2	2	0	1	1	1	0	1	0
$Ni(No_3)_2$ (50 mg/1)	6	4	4	2	2	0	1	1	1	0	0	0
$Ni(NO_3)_2$ (300,000 mg/1)	2	2	2	2	2	2	0	0	1	0	0	0

WS = Whitestore Clay
F = Faceville Clay
H = Hoytville Clay

All fixed-wall permeameter tests were conducted using gradients ranging from 65 to 190 while all the flexible-wall permeameter tests were conducted at a gradient of 200. The turbidity and pH of all the leachates were monitored throughout this project. Atomic adsorption analysis was also conducted on selected clay-chemical combinations.

A major problem in conducting hydraulic conductivity tests is knowing when enough data have been collected to determine if steady state has been reached for the test cells. For this project, readings of permeant liquid inflow level are taken at certain time intervals throughout the test, ranging from 4 to 24 hours. The hydraulic conductivity is computed for each time interval. From this set of data the first ten points are taken and a linear regression analysis is performed to determine the slope of the hydraulic conductivity vs pore volume curve. The first point is then dropped and another value is added on the other end of the pore volume series. The slope is calculated again, and in sequence the method is repeated. In the beginning of the test this slope could be large but as the test progresses it could decrease and approach zero when steady state is obtained.

The criteria for when steady-state is obtained is defined for this research by the following two criterion:

- when the slope of the curve cannot be shown to be significantly different from zero at the 95 percent confidence level and
- at least one pore-volume of the liquid has passed through the sample.

These criterion have proven useful in recent studies conducted at Duke University (2). When the slope of the regression line cannot be shown to be significantly different from zero, the y-intercept value is the equilibrium hydraulic conductivity. For this project all hydraulic conductivity results are reported as y-intercept values or mean y-intercept values when more than one replicate was run.

The determination of the hydraulic conductivities of clays to various permeants can be both time consuming and costly. Because of this, alternative tests are needed which can estimate the effect that inorganic chemicals have on clay particles. In this project the effectiveness of five laboratory tests for predicting the effect inorganic chemicals have on clay particles were evaluated. The five tests include: Specific Resistance to Filtration (SRF), Capillary Suction Time (CST), Settling (S), Plasticity Limits, and Shear Strength.

PROBLEMS

Due to the low water holding capacity of the clays tested, poor results were obtained from the SRF and CST tests. Using standard procedures, these tests are unsuitable for predicting the effects inorganic chemicals have on clay particles.

Approximately fifty fixed and flexible-wall permeameter tests were conducted for this project. Of all these tests only two flexible-wall tests had to be terminated early; one because of a leak in the membrane and the other due to chemical failure of the quick connects in the permeant supply system. In both cases concentrated nickel nitrate (300,000 mg/l) was being used as the permeant.

RESULTS

The hydraulic conductivity results obtained during this research effort can be seen in Table 2. Ninety percent confidence intervals were determined for all the tests in which more than one replicate was performed. From the Table it is seen that no significant changes in hydraulic conductivity occurred in any of the clays upon permeation of the samples with the chemical solutions. Also, the hydraulic conductivity test results obtained from both the fixed and flexible-wall permeameters confirm the repeatability of the data.

The leachate passing through the clay soil samples was also analyzed. The turbidities of all the leachates were very low indicating that no piping occurred in the clay samples. In addition, sample leachates from fixed and flexible-wall tests with Faceville clay were analyzed for metal content using atomic absorption techniques. The analyses indicated that no chemical breakthroughs

TABLE 2. HYDRAULIC CONDUCTIVITY TEST RESULTS
(* 10^{-8} cm/sec)

	FIXED-WALL		FLEXIBLE-WALL	
	SHORT-TERM	LONG TERM	SHORT-TERM	LONG TERM
WHITE STORE CLAY				
0.01 N $CaSO_4$	2.3±0.3	1.6±0.1	2.8±1.6	1.6
500 mg/1 $FeCl_3$	3.7±1.0	6.2±2.0	2.1	ND
50 mg/1 $Ni(NO_3)_2$	2.8±0.6	3.0±0.5	2.0	ND
300,000 mg/1 $Ni(NO_3)_2$	2.2±0.5	ND	ND	ND
HOYTVILLE CLAY				
0.01 N $CaSO_4$	1.0±0.3	1.6±2.8	0.8±0.3	ND
500 mg/1 $FeCl_3$	1.0±0.1	ND	1.3	ND
50 mg/1 $Ni(NO_3)_2$	0.8±0.1	ND	0.9	ND
300,000 mg/1 $Ni(NO_3)_2$	0.8±1.4	ND	ND	ND
FACEVILLE CLAY				
0.01 N $CaSO_4$	3.8±0.8	5.2±1.3	6.3±1.4	ND
500 mg/1 $FeCl_3$	6.4±0.5	8.0±6.9	6.2	ND
50 mg/1 $(Ni(NO_3)_2$	4.3±2.6	11.0±6.0	5.6	ND
300,000 mg/1 $(Ni\ (NO_3)_2$	5.1±1.9	ND	ND	ND

KEY: ND indicates Not Determined

NOTE: Confidence intervals are based on the number of replicates indicated in Table 1.

occurred for the 500 mg/l ferric chloride or the 50 mg/l nickel nitrate solutions. However, chemical breakthrough did occur with the 300,000 mg/l solution of nickel nitrate. No chemical breakthroughs were observed at the lower concentrations since the cation exchange capacities of the clays were not exceeded: several hundred pore volumes of the low concentration liquids would have been necessary to satisfy the CEC's of the clays.

Results from the laboratory tests conducted to estimate the effect of inorganic chemicals on clay particles show that two of the tests, Specific Resistance to Filtration and Capillary Suction Time, do not provide useful results due to the low water holding capacities of the clays. However, while other procedures did produce laboratory results, including settling tests, liquid and plastic limits, the predictive capabilities of these other tests cannot be adequately evaluated since none of the chemicals tested altered the hydraulic conductivities of the clays.

From the results discussed above, the following conclusions were drawn:
1) Ferric chloride and nickel nitrate did not significantly alter the hydraulic conductivities of the clays at the concentrations tested. 2) Screening tests are potentially useful for predicting changes in hydraulic conductivity caused by inorganic chemicals; however, because no significant changes in hydraulic conductivity were observed in this research project the results of the screening tests are not conclusive.

ACKNOWLEDGEMENTS

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EFFECTIVE POROSITY OF GEOLOGIC MATERIALS

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ABSTRACT

The understanding of effective porosity and the development of a method for measuring the effective porosity of geologic materials is essential to predict the rates of ground-water solute movement through those materials. This project is designed to explore the phenomena of effective porosity as a dynamic soil/solute parameter. Modern chromatographic techniques are being adapted as a method for measuring the effective or dynamic porosity of unweathered, unoxidized clay soils at three sites. Preliminary work has isolated some problems associated with collecting "undisturbed" soil samples and conducting tracer experiments on low hydraulic conductivity soils in laboratory columns.

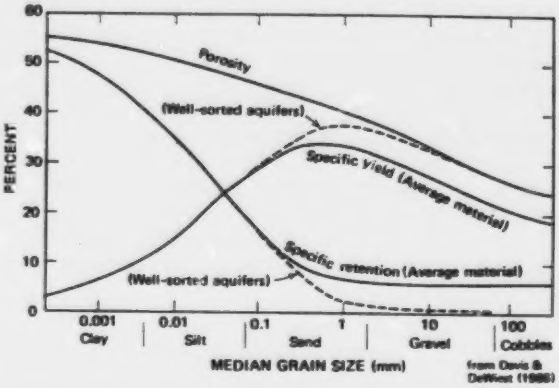
INTRODUCTION

Soil scientists, soils engineers, geologists, and ground-water hydrologists historically have been concerned with the bulk rates of water movement through soils and geologic materials. The application of Darcy's equation has been the traditional method used for making these calculations. With the emergence of ground-water pollution problems, emphasis has shifted towards the prediction of linear rates of solute movement through near surface soils and deeper lying geologic materials. To make these types of predictions accurately, the dynamic porosity of the soils and solutes in question must be known.

For the purposes of this project, dynamic porosity is defined as that portion of the total void space of a soil matrix that is available for fluid flow. Unconnected or dead end pores and some very small but interconnected pores in a soil matrix are not available for fluid transport (1). Similarly, that small portion of fluid (and the void volume it occupies) held on the individual soil grains by interfacial forces is not available for fluid transport (11). Effective porosity can then be considered to be the same as the dynamic porosity of the soil matrix.

The volume of voids not available for fluid flow is a function not only of the soil matrix, but a combination of the soil matrix and the chemical nature of the solution of interest. Fluids containing large molecules in solution may be excluded from pores within the soil matrix that other fluids containing smaller solute molecules would be capable of penetrating (3,15). Similarly, the amount of immobile fluid phase held on the individual soil grains is a function of both the soil particle surface characteristics and those of the fluid.

Effective porosity should not be confused with total porosity, drainable porosity, specific yield, or gravity drainage. Use of any of these parameters as an estimate or substitute for effective porosity can affect the resulting estimates of travel times for solutes through geologic materials by several orders of magnitude. Based on current concepts of effective porosity the use of specific yield or drainable porosity for relative coarse grained soils may be acceptable. The number of "dead end" pores is relatively small and the amount of volume occupied by fluid held in place by surface tension forces is small. Conversely, for fine grained soils the small size of all pores and the large number of "dead end" pores increases the potential impact of both phenomena. Figure 1 illustrates the importance of an accurate determination of effective porosity for fine grained soils. The difference between the total porosity values and specific yield are somewhat comparable to the differences between total porosity and effective porosity. The differences become greater with decreasing soil grain size.



from Davis & DeWiest (1966)

Figure 1. Total porosity relationship to other soil/solute parameters.

Based on the general description above, the measurement of dynamic porosity is most important when attempting to predict the travel times of solutes in clays and other fined grained soils. The estimates of travel times through artificial and natural clay liners is particularly dependent on the ability to determine the dynamic porosity for the anticipated soil/solute combinations. Travel times through thick till and clay sections previously believed to provide "years of protection" to underlying aquifer systems also need to be reevaluated. The variability of dynamic porosity for various classes of chemical compounds and soil combinations is currently unknown.

PURPOSE

The purposes of this project are 1) to develop an understanding of the dynamic porosity phenomena, 2) to develop a technique for measuring the dynamic porosity of fine grained soils, and 3) to attempt to relate dynamic porosity values to other physical soil/solute characteristics. An extensive literature search has been conducted on the subjects of hydrodynamic and physical properties of soil; flow in saturated porous media; tracer movement in saturated porous media; measurement of effective porosity of saturated porous media; and chemical reactions in porous media.

APPROACH

The review of associated technical literature was designed to develop a conceptual understanding of the interaction of soil matrices and various types of solutes. The conceptual understanding of the dynamic porosity phenomena was then used to apply chromatography techniques to address the problems associated with performing effective porosity measurements.

To verify the ability of the selected testing procedure, preliminary tests were conducted on columns packed with glass beads and silica sand. This was designed to minimize the problems associated with soil/solute reactions, grain size and shape, and packing arrangements. Performing tests on column media of known characteristics also permitted the evaluation of the sensitivity of the testing apparatus.

To minimize the effects of secondary porosity, sites for collecting soil samples were chosen where 1) clay soils are present within the upper 50 feet of the unconsolidated materials, 2) the soils are unweathered and unleached, and 3) oxidation has been minimal. These selection criteria provide native samples that most closely approximate the glass bead columns and the medium contained in standard chromatography columns. They also should provide samples that should most closely exhibit only primary or intergranular porosity.

Samples are being collected in a specially constructed split-spoon sampler. The sampler contains a 24-inch length of polycarbonate plastic tubing and is fitted with a cutting shoe sized to match the inside diameter of the tubing, therefore minimizing disturbance to the sample. The polycarbonate tubing is then cut into appropriate lengths, fitted with porous end frits and caps, and used as the columns in the laboratory studies.

Successful tracer studies demand that extremely low flow rates be maintained.

This approach deviates considerably from the techniques routinely applied to laboratory hydraulic conductivity testing. The relative rates of chemical diffusion of the selected tracers and the rate of solute migration through the soil column must be controlled near field conditions to effectively conduct the desired measurements. The resulting long periods required to conduct experiments under hydraulic gradients representative of expected field conditions require the use of special apparatus (see figure 2).

A computerized flow control, sample collection, and data collection system was designed and constructed to operate four columns simultaneously. This permits four simultaneous replicate measurements of soil samples from each site. Integral parts of the system include: a fixed head source, flow measurement accumulators, inlet pressure sensors, tracer injection ports, soil columns, outlet pressure sensors, and solute collection devices. The fixed pressure source is maintained by a regulated supply of a mixture of nitrogen and carbon dioxide. The gas contacts the water in the supply reservoir and in each accumulator. The supply reservoir is capable of supplying water to all our accumulators simultaneously.

The flow measurement accumulators are devices used when very low flow rates are encountered (6). The accumulators in this system are constructed of 2 mL glass pipets enclosed in plexiglass cylinders. Flow rates are determined by measuring the time it takes the water/gas interface in the accumulator to move from an upper to a lower sensing device. Two electric eyes serve as the triggering mechanisms which control the actuation of the three-way valves to refill each accumulator without disturbing the flow through the columns.

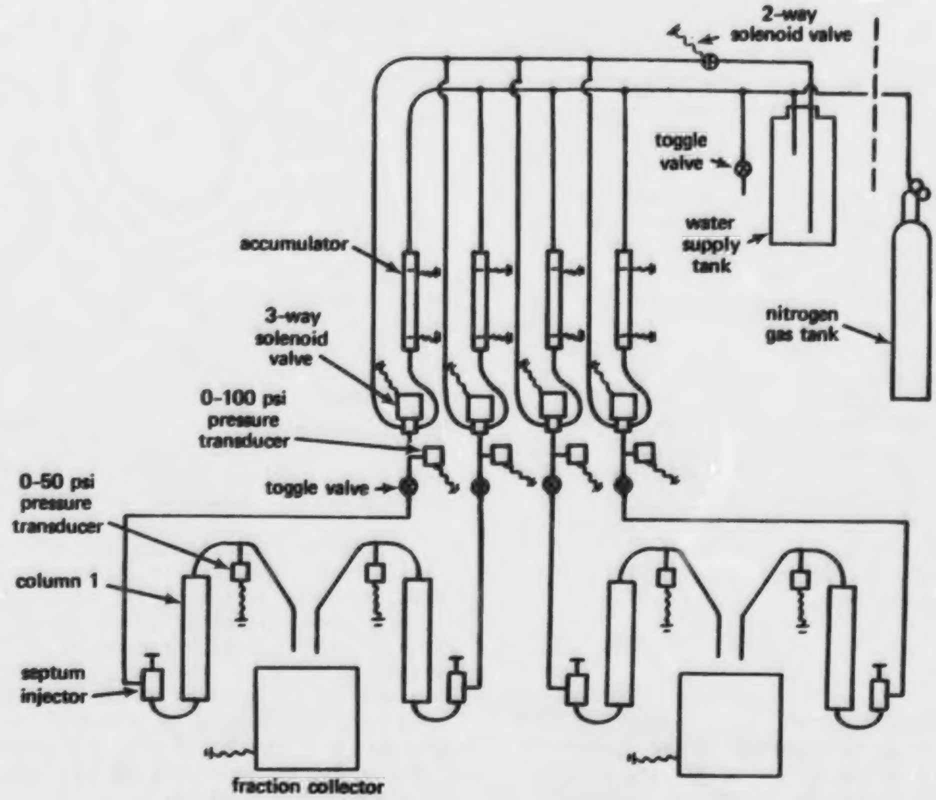

2-way solenoid valve
toggle valve
water supply tank
nitrogen gas tank
accumulator
3-way solenoid valve
0-100 psi pressure transducer
toggle valve
0-50 psi pressure transducer
column 1
septum injector
fraction collector

Figure 2. Laboratory apparatus for hydraulic conductivity and dynamic porosity measurements.

The inlet and outlet column pressure sensors are strain-gage type transducers. The transducers are connected to the flow system with stainless steel fittings designed to minimize dead space. A tracer injection port at the head of each column allows the introduction of small quantities of solute or tracer into the flow system with minimal disturbance of the system.

The soil columns, constructed of polycarbonate plastic, have an inside diameter of 1.25 inches and an outside diameter of 1.50 inches. By maintaining at least a 30-40:1 ratio between the column and particle diameters, local velocity effects, such as channeling or radial velocity gradients, can be effectively minimized (10). The top and bottom of each column (end caps) are constructed of Delrin plastic. The end caps are milled to accept a 5 micrometer pore size stainless steel frit and fittings to connect to the inflow and outflow tubing.

The solute sample collection device is an ISCO Retriever II fraction collector. This device controls the separation of column outflow for analysis, provides a measure of column outflow rate, and signals the computer to record the time when discrete solute samples are collected.

The laboratory apparatus will be used to measure both total and dynamic porosity. To measure total porosity, a nonsorbing, nonreactive tracer similar to the solvent will be used. The close match between molecular dimensions and diffusion coefficients ensures that "all" voids available to the solvent will be penetrated. The resulting retention times for migration of the tracer will enable calculation of total void volume and total porosity. Tritiated water is the tracer to be used for this phase of the experiment.

The use of tritiated water to determine total porosity is believed to be superior to traditional drying, weighing, or bulk density measurement techniques. Drying of soil either by heating or a solvent such as acetone has been shown by some studies to result in alteration of the micro-structural arrangement of soil particles, leading to changes in both total pore space and pore size distribution.

To measure dynamic porosity, a nonsorbing, nonreactive anionic tracer of much larger molecular dimensions will be used. This tracer will penetrate only those voids capable of accomodating the tracer molecules. Penetration of dead end pores or pores smaller than the tracer molecule is not anticipated. The resulting retention time will be used to calculate a dynamic porosity for this particular tracer. A porosity based on elution data is defined as a dynamic porosity (2,7,12,13,14). It should be emphasized that a dynamic porosity measurement for a specific soil sample will be representative only of the tested soil and other solutions containing chemicals with similar characteristics. Additional experiments must be conducted for a variety of soil types and solute combinations to develop a usable dynamic porosity data base. Attempts will be made in this study to select tracers of differing molecular size to further document the utility of molecular exclusion principles to measure effective porosity in clayey soils.

PROBLEMS ENCOUNTERED

The principle problems encountered to date have been the development of the laboratory apparatus. Scaling up chromatographic procedures to the extent required to perform experiments on soil columns has required careful documentation and control of sources of error. Head losses in the testing apparatus and possible chemical interferences must be minimized. Similarly, due to the extremely low flow rates required to conduct this type of experiment, a continuous recording or automated system had to be developed.

Finally, the selection of appropriate tracers cannot be made until the mineralogy of the samples to be tested are well documented.

RESULTS

Work during the first year of this project has centered on the development of the laboratory testing apparatus. Hydraulic conductivity (k) is measured by the constant-head permeameter method (8). Values are automatically calculated, stored and compared to previous values every five seconds which permits the detection of changes in hydraulic conductivity due to oxidation or microbial growth. Leaks in the testing apparatus are also detected by this methodolgy.

The apparatus for collection of soil samples has been constructed and tested. One set of samples has been collected and installed in the laboratory testing apparatus to assure that the field sampling mechanism can be used for the column studies.

Initial tracer experiments using Rhodamine dye were conducted on a temporary apparatus to study the behavior of various sized particles (table 1) in a chromatographic situation. The apparatus consisted of an Altex pump, Model 110 A, in series with a Data Instruments pressure transducer (100 psi), SSI septum injector, column, a second pressure transducer (50 psi), and Gilson Filter Fluorometer, Model 121. Inlet and outlet pressure transducer outputs were monitored on a Cole Palmer dual-pen strip chart recorder. Fluorometric data were collected and stored in a Hewlett-Packard 3390 A Integrator.

TABLE 1. PARTICLE SIZE AND COMPOSITION OF TESTED COLUMNS

Δ	Particle size um	Particle composition	Size Classification USGS	USDA
1	200-500	Ottawa sand	Medium sand	Med. sand/ fine sand
2	106-250	Ottawa sand	Fine sand	Fine sand
3	10-75	Glass beads	Silt	Very fine sand/silt
4	5-20	Glass beads	Silt/ clay	Silt

Four ranges of particle sizes were utilized as the column packing in these experiments.

Rhodamine WT was used as an organic tracer in 1×10^{-3} M $NaHCO_3$ buffered water. Recovery of the tracer was greater than 85% in all cases. All columns were run at a minimum of seven different flow rates in the range of 5 mL/min to 0.1 mL/min. All runs were conducted in duplicate.

Results to date indicated that the columns perform as an effective chromatographic tool. Separation of the Rhodamine WT spike was attained with the expected Gaussian curve observed at greater pore water velocities. However, effluent concentration curves broadened at lower pore water velocities (10^{-3} cm/sec). This phenomenon was expected since diffusive transport is more significant at lower flow velocities. Band broadening is a real-time measure of diffusion processes under controlled flow conditions.

Batch sorption tests were conducted on the four column packing materials. Concentrations tested ranged from 0.119 ppm to 119 ppm. The tests were conducted for a period of eight days. The 10-75 um diameter glass beads were the only material which showed a statistically significant decrease in fluorescence. This trend was noted only at the end of the eight day test. The decrease in fluorescence was observed at three tracer concentrations, 119 ppm, 11.9 ppm, and 0.119 ppm.

Hydraulic conductivities (k) were calculated at every flow rate for each column. A linear increase in k was noted with increasing flow rate to a maximum after which it tailed off (see figure 3). This trend supports Relyea's (13) statement that at high velocities the dynamic pore volume of a sample can decrease. With the diminished pore volume, a rise in pressure is necessary to maintain a constant flow velocity.

The total porosity of all columns was determined statically according to the following:

$$V_v = V_{column} - V_{solid}$$

where: V_v = volume of voids
V_{column} = total volume of column
V_{solid} = total volume of solids

[wt. of solid in column bulk
density of solid]

and $TOT = (V_v/V_{column}) \times 100$

where: TOT = total porosity (%)

By this definition total porosity includes all void space whether interconnected or isolated. This agrees with the use of the term by Dotson et al. (7), Coats and Smith (4), and Corey and Horton (5). Table 2

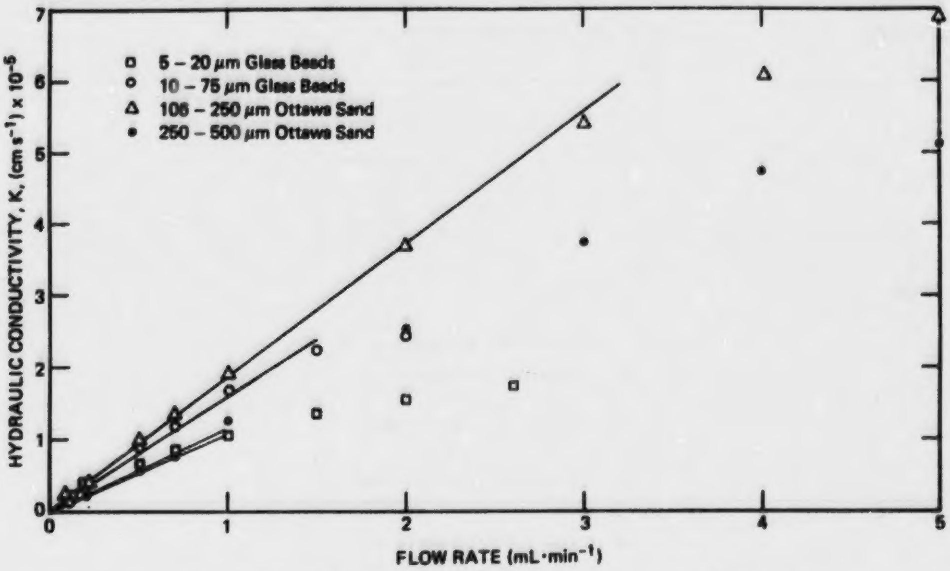


Figure 3. Hydraulic conductivity versus flow rate.

presents the results for the tested columns.

TABLE 2. TOTAL AND DYNAMIC POROSITIES FOR TESTED COLUMNS

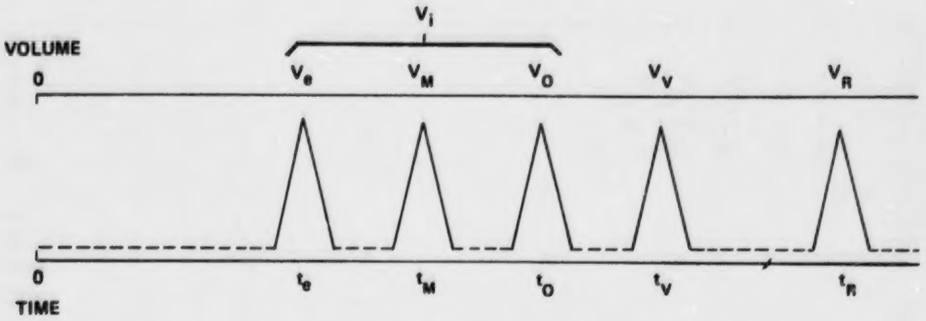
Column no.	V_v void volume (static) (mL)	e_{TOT} total porosity (%)	V_{DYN} pore volume (dynamic) (mL)	n_d dynamic porosity (%)
1	35.6	36.5	38.64±.33	40.0±1.3
2	43.1	45.3	37.68±.28	39.5±0.4
3	37.1	38.4	37.64±.30	39.2±0.7
4	29.1	29.1	30.49±34	34.0±0.3

The values of dynamic porosity are equal to or greater than those calculated for total porosity in these four cases. The results of the batch adsorption studies indicate that retention is not a factor. Therefore, a combination of experimental error, packing, and pore size distribution are the likely explanations. The materials used for Columns 3 and 4 were uniform glass spheres and exhibited little size variability. Consequently, these types of packing yield little, if any, detectable dead-end pore space (9). Columns 1 and 2 were composed of Ottawa sand, two to five hundred times greater in particle diameter than the glass beads. This large grain size and the decrease in both sphericity and roundness contributed to a porosity which is equal to the total porosity within experimental error.

Horvath et al. (12) defined three different hold-up times t_e, t_m, and t_o, and corresponding hold-up volumes, $V_e \cdot V_m$, and V_o for a variety of solutes in a mobile phase. These factors will be evaluated by varying the size of the tracer in a series of experiments and are graphically illustrated in figure 4.

With respect to the preliminary experiments for Columns 1 and 2, the chromatographic volume equals the volume of voids, $V_o = V_v$. Therefore, all voids within the columns participate in fluid flow. The large grain size, the narrow grain size distribution, and the lack of both sphericity and roundness of particles are all factors contributing to the equivalency of dynamic and static void volumes. Thus, the unsorbed solute, Rhodamine WT, explored mainly the interstitial volume, V_e. The intrastitial volume, V_i, was very small.

Columns 3 and 4 are composed of nonporous glass beads, a column material impervious to solute and allowing no intra-



where V_e – interstitial fluid volume
V_i – intrastitial fluid volume
$V_i = V_O - V_e$
V_M – unabsorbed solute volume
$V_M = V_e + \psi V_i$
V_O – chromatographic hold-up volume
V_V – void volume (statically determined)
V_R – volume of a retarded solute
ψ – fraction of intraparticulate fluid space accessible to solute

Figure 4. Schematic illustration of chromatographic volumes.

stitial fluid volume, i.e. $V_i = 0$. The use of Rhodamine WT therefore provides an exact indication of interstitial volume, V_e. Again, in these two columns V_O is coincidental with V_e (i.e. $V_O = V_e = V_V$).

The soils chosen for this project are to be fine grained with a high percentage of clay. Expected hydraulic conductivities (1×10^{-7} cm/sec) will be a hundred times lower than those observed with the four columns tested to date. Resulting porosities, therefore, are expected to have considerable intrastitial volumes. Dead-end pore volumes also will become a recognizable factor with these pore and particle size distributions over a range of flow rates. The chromatographic hold-up volume, V_O, will be measured by use of a conservative tracer, HTO. A second tracer of larger molecular dimension, i.e. Rhodamine WT or polyamino acid, will be used to determine the minimum interstitial volume available for flow. This information will provide porosity values as defined by the molecular dimension of the the two tracer molecules.

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SOLUBILITY PARAMETERS FOR PREDICTING MEMBRANE-WASTE LIQUID COMPATIBILITY

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ABSTRACT

The swelling by absorption of waste liquids is a major factor in the loss of properties that can affect the performance and service lives of polymeric membrane liners. Swelling can result in softening, loss of mechanical strength and elongation, an increase in permeability, increased tendency to creep under load, and greater susceptibility to polymer degradation. The significant factors involved in the swelling of a polymeric composition in a liquid are:

- Solubility parameter or cohesive energy density of the polymer relative to that of the liquid.
- Crosslinking of the polymer.
- Crystallinity of the polymer.
- Filler, plasticizer, and soluble constituents in the total compound.

Of these, the solubility parameter is probably the most significant factor and the subject of this research project.

Most of the dissolved organic constituents in a waste liquid have solubility parameters in the range of 8-10, which is the same range as the polymers used in the manufacture of the membranes. Thus, in order to avoid excessive swelling of a liner by a given waste liquid, it is desirable to select a polymeric membrane that has a solubility parameter outside this range or that differs in solubility parameters from the organic constituents in the waste.

This paper presents an analysis of data from the literature with respect to the solubility parameters of organic solvents in liquids and of polymeric materials; it also presents the results of laboratory testing of selected commercial lining materials in various organic solvents having different solubility parameters.

INTRODUCTION

The compatibility of a proposed polymeric membrane liner with the waste to be contained is one of the principal requirements in permitting the design and construction of a waste management facility. A liner material must be of low permeability and must maintain its integrity to effectively contain the particular waste for the required length of time; that is, it must be compatible with the waste liquid. In some cases, this requirement will be for extended periods of time, e.g. as in landfills; in others, the requirement may only be for a few years, e.g. in an evaporation pond.

The technology involving the use of polymeric membranes for lining waste management facilities is relatively new, and present information in the open literature regarding the compatibility of liners [illegible] waste liquids is limited. Because waste liquids generally are highly complex mixtures with the possibility of detrimental effects due to the combination of components, the compatibility of a lining material with a specific waste liquid should be assessed by immersion testing, such as the EPA Test Method 9090 (EPA, 1984). In this method samples of the candidate membranes are immersed in representative samples of the waste for up to 4 months, and the properties of the membranes are measured as a function of time. Changes in the membrane and loss of values of selected properties are indicative of noncompatibility. Specific protocols need to be developed to assess long-term effects.

In assessing the suitability of a polymeric composition for a product designed for service in contact with liquids and chemicals, it is common practice in the rubber and plastics industries (Beerbower et al, 1967) to use solubility parameters in designing or selecting the polymer compound. The compound is then exposed to the liquid with which it is to be in contact and the effects of the exposure upon the polymeric composition are measured as a function of time. The swelling and dimensional changes that take place and the changes in mechanical properties, such as tensile strength, elongation at break, tear resistance, and modulus, are normally measured. For many applications the service life will be measured as the time to a definite percentage change or loss in a property relevant to its performance. These criteria are correlated with failures in specific products in simulated service tests or in actual service. In the case of polymeric membranes used as liners, the correlation of failure with changes in specific properties has not been developed. Among the various properties, the swelling or loss in weight of a membrane liner in service appears to be one of the most useful tools in assessing its compatibility with wastes. Associated with these changes in weight are changes in most of the physical properties, such as tensile strength, elongation at break, tear resistance, modulus, hardness, and permeability.

The comparison of solubility parameters of a polymeric membrane liner with that of a waste liquid and its constituents has been suggested as a predictive measure of the swelling that might take place in a membrane liner [Haxo 1981; Matrecon, 1983 (p. 177)]. Applicability of the solubility parameters concept to estimating service lives of liners, however, has not been quantified; this is one of the principal objectives of this project.

This paper is a preliminary report on the work that is being conducted in a study of the possible use of solubility parameters to predict membrane-waste chemical compatibility and service life. It includes presentation of pertinent data from the literature and the results of experimental work which show the relevance of this concept in predicting long-term compatibility of polymeric lining materials to waste liquids.

OBJECTIVES

The principal objectives of the project are:

1. To develop a method of predicting the long-term compatibility of membrane lining materials with different hazardous wastes and hazardous materials and estimating service lives under different conditions.

2. To explore the use of solubility parameters and other indexes that have been used in the polymer industry to select polymer compositions and design products that are in contact with different chemical species, e.g. oils, fuels, lubricants, and other fluids.

Though the principal thrust of this project is the determination of the feasibility of using solubility parameters as a means of determining the swelling and the ultimate service life of polymeric membranes, other factors will be investigated as they probably will be incorporated into the model for estimating service lives of membranes under different service conditions.

TECHNICAL APPROACH

Waste Liquids in Contact with Liners

This project is concerned principally with polymeric membranes in contact with waste liquids that contain organics, either dissolved in water or in mixtures of organics. Two conditions that would be encountered by a liner in contact with these wastes are schematically illustrated in Figure 1.

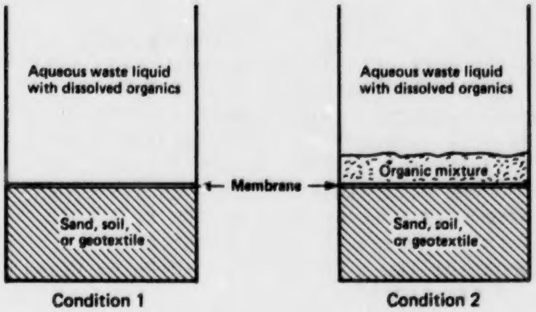
Aqueous waste liquid with dissolved organics
Aqueous waste liquid with dissolved organics
Organic mixture
Membrane
Sand, soil, or geotextile
Sand, soil, or geotextile
Condition 1
Condition 2

Figure 1. Two conditions that membranes could encounter when in contact with waste liquids as liners for waste management facilities.

In the first condition, the waste liquid that contacts the liner consists of water with dissolved organics and probably some inorganics. Though inorganics in a waste liquid may affect organic solubility, it is the organic constituents of the waste liquid that are of importance as they are potentially soluble in the polymeric membrane. The ultimate absorption of an organic chemical by the membrane liner is determined by the relative solubility characteristics of the organic chemicals and the liner. Once the organic chemical, either volatile or nonvolatile, has dissolved in the liner it will diffuse into and swell the liner. If the organic is a volatile material, it can evaporate into the pores of the soil on the downstream side.

In the second condition, the liner would be in direct contact with organic mixtures. Such a condition has been encountered in the field when the organic solvent concentrations having a higher specific gravity than the waste liquid exceed their solubility limit in water and collect on the liner. In this case, the layer would consist of a mixture of several organics which can partition into the lining material. The concentration of organics is considerably higher in the second condition and, consequently, the degree of swelling can be higher and subsequent transmission should be greater.

This project addresses primarily the matter of solubility of organics in lining materials as determined by their relative solubility parameters. The question of dilute solutions of organics and partitioning is also considered in the study of dilute solutions of organics in water and in mixtures of organics.

Swelling of a Polymeric Membrane on Exposure to Waste Liquids

Though most membranes during exposure to an organic waste absorb constituents of the waste liquids and swell, highly plasticized membrane compositions can lose plasticizer and shrink. These two processes can operate simultaneously so that, in some cases, the plasticizer can be extracted from the membrane and, simultaneously, the organic constituents from a waste can be absorbed by the membrane to result in either a net swelling or loss.

The compatibility testing of membrane liner materials for waste management facilities, e.g. EPA Test Method 9090 (EPA, 1984), has been concerned principally with the effect upon liners of immersion in the waste liquid or leachate which is to be impounded and the amount of swelling that takes place. Swelling can result in deterioration of many of the properties of polymeric membranes.

It should be noted, however, that properties of polymeric membranes other than tensile strength, elongation at break, tear resistance, puncture resistance, and gain in permeability can be affected by solvents and waste liquids without showing much swelling. Of particular importance are the effects on semicrystalline membranes which, depending on the polymer, under simultaneous exposure to waste liquids and mechanical stress, can be subject to environmental stress-cracking and rupture.

The major factors involved in the swelling of polymeric membrane compounds in a liquid are:

- Solubility parameter of the polymer.

- Crosslinking of the polymer.
- Crystallinity of the polymer.
- Filler content of the compound.
- Plasticizer content of the compound.
- Soluble constituents in the compound.

The solubility parameter is probably the most important factor and, as noted above, is used by polymer scientists to measure the compatibility of a polymer with a liquid with which it may be in contact.

Crosslinking of a polymer or a rubber reduces its ability to swell in a liquid. The amount of swelling of a crosslinked polymer in a good solvent for the raw polymer can be used as a measure of the degree of crosslinking: the greater the crosslinking, the less the swelling.

Crystallinity of a polymer acts much like crosslinking to reduce the ability of a polymer to dissolve. Highly crystalline polymers, such as high-density polyethylene, will not dissolve in gasoline, even though they are both hydrocarbons and basically similar.

Two factors in liner composition that also contribute to swelling are the amount of filler used in the recipe, e.g. carbon black, silica, or clay, and the amount of plasticizer. The latter is extractable by organic solvents, but only slightly extractable by water.

Generally, rubber and plastic compounds contain minor amounts of water soluble inorganic salts. They enter the compound via the polymer itself, e.g. catalyst traces, salt used in flocculation, etc, and via small amounts in the various compounding ingredients, e.g. many of the non-black fillers contain small amounts of water-soluble constituents. These water-soluble salts can cause swelling by diffusion of water into the mass by the driving force of osmosis.

Solubility Parameters

Solubility parameters have found wide use in determining the solubility of polymeric materials in various solvents. Some of the many applications are reviewed by Barton (1975), such as in the paint and coatings industry. Also of particular interest are the uses of this parameter in studying the plasticization of polymers, in preparing rubber blends, and in the designing of rubber and plastic compositions for contact with various oils, hydraulic fluids, and gasoline (Beerbower et al, 1964, 1967). The latter application is similar to that used in assessing the swelling and performance characteristics of liners that might be exposed to various wastes containing organics.

The solubility parameter (δ) and cohesive energy density (C.E.D) are concepts related by the following equation (Hansen, 1967, p. 104):

$$\delta = (\mathrm{C.E.D.})^{1/2} = (\Delta E/V_m)^{1/2} \qquad (1)$$

where

ΔE = the energy required to vaporize one mole of material

V_m = the molar volume.

Thus, δ is a measure of the potential energy of any material with respect to its energy in an entirely disassociated form, that is, free of any inter-molecular interactions. Intuitively, two different solvents of exactly equal potential energies should be mutually miscible in all proportions with no loss or gain of energy. This model of solubility, termed the solubility parameter model, was developed by Hildebrand (Hildebrand and Scott, 1950, p. 119) and may be expressed by the equation:

$$\Delta E_{mix} = X_1 X_2 (\delta_1 - \delta_2)^2 \qquad (2)$$

where

ΔE_{mix} = the energy of mixing

$X_{1,2}$ = the volume fractions of components 1 and 2

$\delta_{1,2}$ = the solubility parameters of components 1 and 2.

Clearly, the mathematical model agrees with intuition in concluding that equal solubility parameters imply no energy change on mixing.

The potential energy of solvents may be simply expressed as δ, but is in fact a sum of energies due to several different types of molecular interaction. These include dipole-dipole interactions, London dispersion forces, hydrogen-bonding effects, and at very close distances, repulsive effects. These energies are approximately additive [Hildebrand and Scott, 1950 (p. 56); Gardon and Teas, 1976 (p. 428); Hansen, 1967 (p. 104)]:

$$E_{total} = E_1 + E_2 + E_2 + \ldots \qquad (3)$$

It follows from equation (1) that:

$$\delta^2_{total} = \delta^2_1 + \delta^2_2 + \delta^2_3 + \ldots \qquad (4)$$

Consideration of the individual contributions of the solubility parameter components becomes quite important in determining the solubility of complex systems such as polymers. These do not behave in the "ideal" manner assumed in construction of the solubility parameter model and consequently solubility is sensitive to variations in the component solubility parameters, not just the overall solubility parameters.

In order to properly describe the solubility of polymers, models more complex than Hildebrand's solubility parameter model are required. The most important model of this general form was proposed by Hansen (1967) and is termed the three-dimensional solubility parameter model. It is written as:

$$\delta^2_{total} = \delta^2_d + \delta^2_p + \delta^2_h \qquad (5)$$

where

δ_{total} = the Hildebrand solubility parameter

δ_d = the contribution to the total solubility parameter due to intermolecular London dispersion forces

δ_p = the contribution due to intermolecular dipole interactions

δ_h = the contribution due to intermolecular hydrogen-bonding.

Approaches taken by various researchers are described in Beerbower et al (1964), Gardon and Teas (1976), and Van Krevelen and Hoftyzer (1976). A generally useful model will probably require parameters defining polymer crosslinking and crystallinity as well as polymer solubility parameters, and may well not be amenable to a simple graphic presentation.

Type and Composition of Polymeric Membranes

Polymers used in the manufacture of lining materials include rubbers and plastics differing in basic chemical composition and structure, solubility characteristics, chemical resistance, and the mode in which they deteriorate and ultimately fail. These polymers can be classified into four general types:

- Rubbers (elastomers) that are generally crosslinked.
- Thermoplastic elastomers that are not crosslinked.
- Thermoplastics that are generally not crosslinked.
- Thermoplastics that have a relatively high crystalline content, such as the polyethylenes.

Table 1 lists the types of polymers that are used in the manufacture of membrane lining materials and shows the abbreviations used in this paper. The polymers most frequently used in liners have been PVC, CSPE, CPE, IIR, EPDM, CR, and HDPE. The thickness of commercial polymeric membranes for liners ranges from 0.5 to 3.0 mm (20-120 mil), with most in the 0.75 to 1.5 mm (30-60 mil) range.

Most polymeric membranes are based on single polymers, but blends of two or more polymers are being developed and used in liners. Also, different grades (varying in molecular weight, branching, or crosslinking) of a given type of polymer can be used. Such variations can affect the solubility characteristics of the membrane. Generic classifications based on individual polymers have become increasingly difficult, even though one polymer may predominate. All polymers are compounded with auxiliary ingredients which serve different purposes and may also significantly affect solubility characteristics as well as permeability and other physical properties.

TABLE 1. POLYMERIC MATERIALS USED IN LINERS

Polymer	Abbreviation	Type of compound[a]	Thickness, mm	Fabric reinforced
Butyl rubber	IIR	XL	0.9-2.3	Yes, no
Chlorinated polyethylene	CPE	TP	0.5-1.0	Yes, no
Chlorosulfonated polyethylene	CSPE	TP,XL	0.6-1.0	Yes, no
Elasticized polyolefin	ELPO	CX	0.6	No
Elasticized polyvinyl chloride	PVC-E	TP	0.8-0.9	Yes
Epichlorohydrin rubber	ECO	XL	1.67	Yes, no
Ethylene propylene rubber	EPDM	XL,TP	0.5-1.6	Yes, no
Neoprene	CR	XL	0.5-1.8	Yes, no
Nitrile rubber	NBR	XL	0.8	Yes
Polyester elastomer	PEEL	CX	0.18	No
Polyethylene,				
low-density	LDPE	CX	0.3-0.8	No
linear low-density	LLDPE	CX	0.5-0.8	No
high-density	HDPE	CX	0.5-3.1	No
Polyvinyl chloride	PVC	TP	0.3-0.9	Yes, no
Polyvinyl chloride oil resistant	PVC-OR	TP	0.8	Yes, no
Thermoplastic elastomer	TPE	CS	1.0	No

[a] TP = Thermoplastic; CX = partially crystalline; XL = crosslinked.

The basic compositions of the different types of compounds are shown in Table 2. The crosslinked compositions are usually the most complex because they contain a crosslinking system. Thermoplastics, except for CSPE compounds, contain no curatives; although thermoplastic as supplied, CSPE liners contain crosslinking agents that allow the polymer to crosslink during service. Liners based on crystalline polymers have the simplest composition and generally consist of the polymer, a small amount of carbon black for ultraviolet protection, and antidegradants.

Several of the components of a liner formulation can be affected during service when the liner is either immersed in the liquid or exposed to the weather. Low molecular weight fractions in the original polymer or blend can be lost. The oils and plasticizers are potentially extractable and, in some cases, biodegradable; antidegradants can be extracted. Loss or change

TABLE 2. BASIC COMPOSITIONS OF POLYMERIC MEMBRANE LINER COMPOUNDS

	Composition of compound type, parts by weight		
Component	Crosslinked	Thermoplastic	Crystalline
Polymer or alloy	100	100	100
Oil or plasticizer	5-40	5-55	0-10
Fillers:			
Carbon black	5-40	5-40	2-5
Inorganics	5-40	5-40	...
Antidegradants	1-2	1-2	1
Crosslinking system:			
Inorganic	5-9	a	...
Sulfur	5-9	...	...

[a] An inorganic curing system that crosslinks over time is incorporated in CSPE liner compounds.

in any of these components can affect properties and, thus, compatibility or durability of the compound.

Most of the polymeric membrane liners currently manufactured are thermoplastic. Even if the polymer in the crosslinked form is more chemically resistant (such as CPE and CSPE), it is generally supplied as a thermoplastic because it is easier to make reliable seams and repairs in the field. Membranes based on thermoplastic polymers can be heat-sealed or seamed with a solvent or bodied solvent. Semicrystalline sheetings are generally seamed by various thermal welding or fusion methods.

RESULTS

Solubility Parameters of Solvents and Other Organic Chemicals

Comprehensive tabulations of solubility parameters for common solvents and other organic chemicals have been made by Barton (1975, 1983). Many more data are available in other references, and discrepancies exist among the data for a given solvent. A review and analysis of these data are being made in the performance of this project and the most reasonable values will be selected for use. Typical solubility parameter values for some common solvents of different chemical types are presented in Table 3.

Solubility Parameters of Polymers Used in Manufacture of Membrane Linings

The solubility parameter of a polymer is generally measured by immersing specimens in solvents of different solubility parameter values and observing the swelling or the dissolution of the polymer when the parameters of the two are equal, as illustrated in Figure 2. Most of the data presented in Table 4 for different polymers were obtained in this manner on pure or slightly crosslinked compositions.

Comparison of the Solubility Parameters of Common Solvents and Polymers

Figure 3 shows the solubility parameters of common solvents and some of the polymers that are used or potentially useful in the manufacture of lining materials. A distribution of the solubility parameters of solvents is also shown by the type of solvent. These data show that a high proportion of the common solvents and polymers fall within a solubility parameter range of 8-10. This indicates that a large degree of swelling would take place when polymers are in contact with waste liquids that contain these solvents and possibly dissolution of the polymer will occur.

Swelling of 12 Polymeric Membranes in Solvents of Different Solubility Parameters

In a series of preliminary experiments, a variety of membranes were immersed in a series of 9 solvents, differing in solubility parameters (6.8-14.5). The results of these experiments are presented in Table 5. The results generally show that, for each material, there is essentially a swelling maximum. However, the data are scattered and precise solubility paramater values cannot be set. Additional solvents need to be used in the testing to narrow the values obtained. There is a substantial difference among the polymers in different solvents, with the crystalline type materials showing significantly lower swelling. Since some of the liner compositions, e.g. PVC, contain significant levels of extractables as plasticizers, data for these materials show a loss in weight due to the extraction of the plasticizer.

Swelling of Laboratory-Mixed Compounds of Known Compositions

In view of the possibility of plasticizers and fillers having effects upon the solubility parameters and the fact that some of the liners are probably blends, a series of five polymers were compounded in simple compositions without plasticizer or filler. These polymers included a butyl rubber, a CSPE, an EPDM, a neoprene, and a nitrile rubber. All were lightly crosslinked, except the CSPE compound (Table 6).

Specimens of these compounds were immersed in the same series of solvents as used with the membrane liners discussed above. The results for 28-day immersions, presented in Figure 4, show curves similar to those in Figure 2, except there are insufficient data points. They also show that the CSPE composition disintegrated in the solvents of solubility parameters of 8.8, 9.8, and 10.4. These data are based upon solubility parameters calculated from cohesive energy densities.

TABLE 3. SOLUBILITY PARAMETER VALUES[a] FOR SELECTED SOLVENTS

Name	δ, (cal cm^{-3})$^{1/2}$				
	δ	δ_t	δ_d	δ_p	δ_h
Alkanes					
n-Octane	7.6	7.6	7.6	0.0	0.0
Cyclohexane	8.2	8.2	8.2	0.0	0.1
Aromatic hydrocarbons					
Toluene	8.9	8.9	8.8	0.7	1.0
o-Xylene	8.8	8.8	8.7	0.5	1.5
Tetrahydronaphthalene	9.5	9.8	9.6	1.0	1.4
Halohydrocarbons					
1,1-Dichloroethylene	9.1	9.2	8.3	3.3	2.2
Ethylene dichloride	9.8	10.2	9.3	3.6	2.0
Carbon tetrachloride	8.6	8.7	8.7	0.0	0.3
Tetrachloroethylene	9.3	9.9	9.3	3.2	1.4
1,1,2,2-Tetrachloroethane	9.7	10.6	9.2	2.5	4.6
Ethers					
Epichlorohydrin	11.0	10.7	9.3	5.0	1.8
Tetrahydrofuran	9.1	9.5	8.2	2.8	3.9
Ketones					
Acetone	9.9	9.8	7.6	5.1	3.4
Methyl ethyl ketone	9.3	9.3	7.8	4.4	2.5
Diethyl ketone	8.8	8.9	7.7	3.7	2.3
Aldehydes					
Furfural	11.2	11.9	9.1	7.3	2.5
Esters					
Ethyl acetate	9.1	8.9	7.7	2.6	3.5
Di-n-butyl phthalate	9.3	9.9	8.7	4.2	2.0
Tributyl phosphate	8.8	8.9	8.0	3.1	2.1
Tricresyl phosphate	8.4	11.3	9.3	6.0	2.2
Dioctyl phthalate	7.9	8.9	8.1	3.4	1.5
Nitrogen compounds					
Pyridine	10.7	10.7	9.3	4.3	2.9
Quinoline	10.8	10.8	9.5	3.4	3.7
Formamide	19.2	17.9	8.4	12.8	9.3
Sulfur compound					
Carbon disulfide	10.0	9.9	5.3	8.1	2.1
Monohydric alcohol					
Ethanol	12.7	13.0	7.7	4.3	9.5
1-Propanol	11.9	12.0	7.8	3.3	8.5
1-Octanol	10.3	10.3	8.3	1.6	5.8
Carboxylic acid					
Acetic acid	10.1	10.5	7.1	3.9	6.6
Phenol					
m-Cresol	10.2	11.1	8.8	2.5	6.3
Polyhydric alcohol					
Ethylene glycol	14.6	16.1	8.3	5.4	12.7
Water	23.4	23.4	7.6	7.8	20.7

[a] 25°C solubility parameters: δ = Hildebrand parameter; Hansen parameters: δ_t = total; δ_d = dispersive; δ_p = polar; δ_h = hydrogen-bonding. Metric unit MPa$^{1/2}$ = 2.045 x (cal cm^{-3})$^{1/2}$.

Source: Barton (1975).

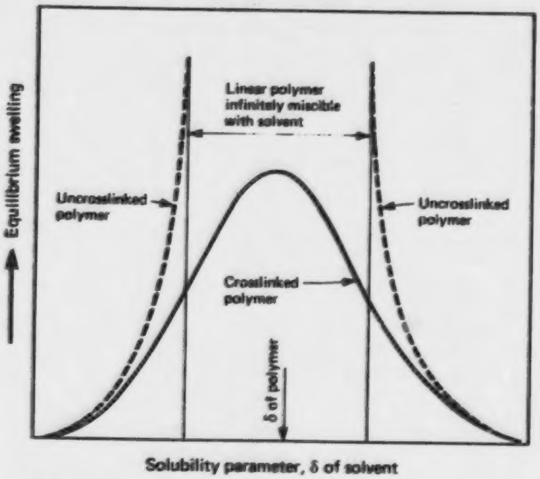

Figure 2. Equilibrium swelling as a function of the solubility parameter of the solvent for a linear (uncrosslinked) and network (crosslinked) polymer [from Van Krevelen and Hoftyzer (1976)].

Swelling of Membranes Based on Different Polymers in Dilute Aqueous Solution of an Organic Chemical

As indicated previously, the most probable condition that liners will encounter is a dilute aqueous solution in which organics are dissolved constituents. In this case, it is necessary for the organic chemicals that are solutes in the water to partition with the polymeric membrane. This problem is similar to that encountered in the partitioning of nondegradable toxic organics dissolved in water that enter the biota and the food chain. In that case, water-octanol partitioning coefficients have become very useful (Leo et al, 1971), and are being explored as they might be applied to dissolved organic waste constituents and polymeric membrane liners.

Partitioning coefficients for dissolved organics between water and polymeric membrane materials have not been located in the literature. Matrecon made an initial study (Haxo et al, 1984) of immersing

TABLE 4. SOLUBILITY PARAMETERS[a] OF VARIOUS POLYMERS[b]

Polymer	Average	Range	Number of determinations
Butadiene/acrylonitrile, 75/25	9.38	9.25-9.50	4
Butyl rubber	7.84	7.70-8.05	5
Ethylene propylene rubber	7.95	7.90-8.00	2
Neoprene	8.85	8.18-9.38	4
Polyethylene	7.94	7.87-8.10	4
Polytetrafluoroethylene	6.2	...	1
Polypropylene	8.1	...	1
Polyvinyl chloride	9.57	9.48-9.70	3
Chlorinated rubber[c]	...	8.5-10.6	...
Chlorosulfonated polyethylene[c]	...	8.1-9.6	...
Polyvinylidene chloride	12.4	12.2-12.6	2
Silicone rubber	7.4	7.3-7.55[d]	2

[a] (cal cm^{-3})$^{1/2}$.

[b] Source - Sheehan and Bisio (1966).

[c] Source - Burrell (1975).

[d] Source - Hauser et al (1956).

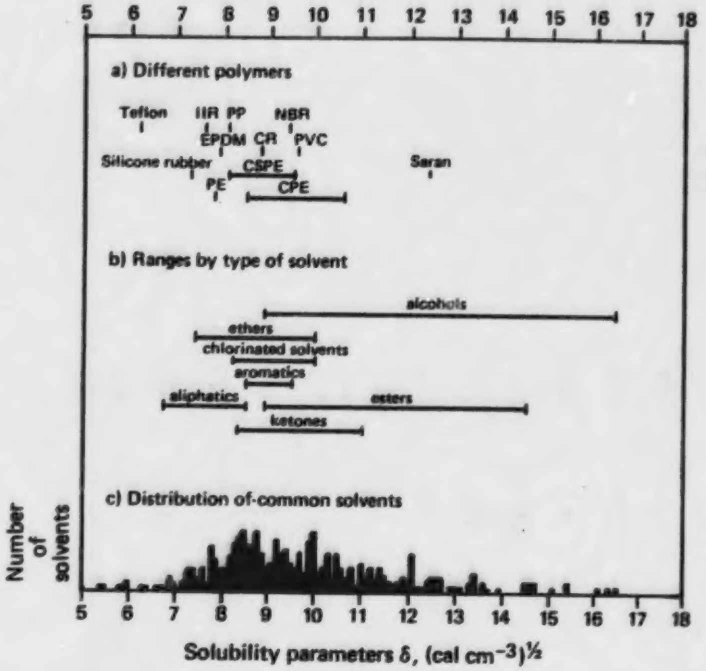

Figure 3. Solubility parameters of common solvents and polymers (data from Barton, 1975, and Burrell, 1975).

a variety lining materials in a dilute solution of tributyl phosphate (TBP). This organic liquid has reported solubility parameters ranging from 8.5 to 9.0 and a solubility in water of less than 0.1%. Preliminary data from this study are presented in Table 7. The data show that, on long exposures, though the concentration of TBP in the water is low, the absorption of TBP and possibly some water by the membranes can reach significant values in which the properties are greatly deteriorated. The data also show that polymers that have solubility parameters differing from the TBP swell less than those that have solubility parameters in the range of 8.5-9.0. Also, the CPE that was crosslinked swelled much less than the thermoplastic CPE. Those membranes that were semicrystalline swelled the least. Such results indicate that factors in addition to solubility parameters must be considered in estimating swelling and ultimately estimating service life.

Immersion of the same liners in 100% TBP resulted in high absorption within a day and a disintegration of PVC and CPE membranes within 14 days, as shown in Table 8. The results of the long-term exposures at 0.1% concentration and the short-term at several days follow the same trends, but the magnitudes differ greatly in some cases; for some liners there were losses in weight probably due to plasticizer extraction. The data indicate that there are differences in partitioning coefficients at different concentrations and some solubility of the plasticizers in the water.

A proposed study in this project, in which samples of liners will be immersed in dilute aqueous solutions of organics and tested, should clarify these discrepancies. This study should also yield information with respect to the partitioning of dissolved organics between the solutions and various polymeric membrane liners.

TABLE 5. SWELLING OF MEMBRANES AFTER 28 DAYS OF IMMERSION IN VARIOUS ORGANIC CHEMICALS AT ROOM TEMPERATURE

Membranes			Solvent and solubility parameter, % weight increase									
Polymer[a]	Type	Extractables, %	Iso-octane 6.8	Normal octane 7.6	Cyclohexane 8.2	Xylene 8.8	Acetone 9.8	Tetralin 9.8	Tetrachloroethane 10.4	Normal propyl alcohol 11.9	Methyl alcohol 14.5	DI water 23.2
IIR (44)	XL	11.79	64	79	133	102	-4	136	109	-4	-1	1[b]
CPE (100)	XL	17.42	-10	-10	5	100	-2	149	294	-9	-7	1[c]
CPE (154)	TP	11.57	2	3	18	89	46	159	>500[d]	1	2	4[e]
CSPE (148)	TP	6.17	2	4	108	>200[d]	4	>700[d]	>1000[d]	-2	6	5[e]
ECO (178)	XL	7.63	70	85	124	114	-3	160	120	-3	-1	28[e]
ELPO (172)	CX	4.01	27	34	67	61	2	59	46	3	1	0.4[e]
EPDM (94)	TP	14.1	-2	0	5	44	6	86	6	-5	-8	...
HDPE (184)	CX	0.73	5	6	8	8	1	9	9	0	0	...
HDPE-A (181)	CX	2.09	12	14	27	23	1	22	20	0	0	...
PB (221)	CX	3.68	...	19	47	22	...	...	28	...	...	...
PVC (150)	TP	32.73	1	0	4	8	83	60	392	-4	-8	3[e]
PVC-OR (144)	TP	30.97	-3	-1	1	2	117	37	492	-8	-13	4[e]

[a] Matrecon liner serial number in parentheses.

[b] 174 days.

[c] 49 days.

[d] Specimen too deteriorated to weigh.

[e] 35 days.

TABLE 6. POLYMER COMPOSITIONS TESTED FOR SOLUBILITY PARAMETERS

Ingredient		Polymer (parts by weight)				
Generic name	Trade name	Butyl	CSPE	EPDM	Neoprene	Nitrile
Butyl rubber	Polysar 301[a]	100	...	...	...	...
Chlorosulfonated polyethylene	Hypalon 45[b]	...	100	...	...	...
Ethylene propylene rubber	Nordel 1040[b]	...	...	100	...	...
Neoprene	Neoprene W[b]	...	...	...	100	...
Nitrile rubber	Hycar 1042[c]	...	...	...	...	100
Zinc oxide	Protox 168	3	...	3	5.0	5.0
Magnesium oxide	Maglite D	...	4.0	...	4.0	...
Stearic acid	F300	2.5	...	2.5	...	1.0
MBT	Captax	0.75	...	0.75	...	...
TMTDS	Tuex	1.5	...	1.5	...	3.5
Sulfur	...	1.25	...	1.25	...	...
Antioxidant	Flexamine	...	...	...	1.0	...
Accelerator	Pennzone E	...	...	...	0.5	...
Wax	Carbowax 4000	...	1.5	...	...	...
Molding conditions:						
Temp., °C	...	160	160	160	160	160
Time, min	...	40	d	40	30	30

[a] Polysar Corporation.

[b] E.I. dupont de Nemours and Company, Inc.

[c] B.F. Goodrich Company.

[d] Molded and cooled in press; the others were crosslinked.

In the second condition that a liner might encounter in service (shown in Figure 1, Condition 2), in which mixtures of organics are in contact with liners, somewhat similar conditions to those with organic mixtures probably would exist when the minor organic constituents in the waste are nonvolatile and have different solubilities in the organic mixture and in the liner. The major organic constituent of the mixture will tend to be absorbed in the liner and the minor constituents will partition between the organic mixture and the polymeric membrane.

In the case of membranes based on plasticized compositions, the organic solvent layer above the membrane will probably extract plasticizers such as occurred with the liners immersed in 100% TBP. If the liner is a plasticized PVC composition, loss of plasticizer will cause the membrane to stiffen and become brittle. These parameters will be measured during the project.

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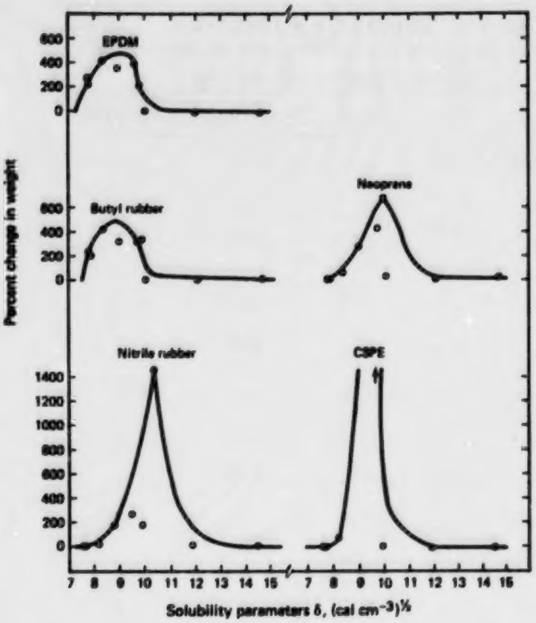

Figure 4. Swelling of five lightly cross-linked nonfilled, polymeric compositions in nine solvents of different solubility parameters.

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TABLE 7. EFFECTS ON PROPERTIES OF MEMBRANES[a] ON IMMERSION IN A DILUTE[b] AQUEOUS SOLUTION OF TRIBUTYL PHOSPHATE FOR 522 DAYS

Polymer	Butyl	CPE	CPE	CSPE	ELPO[c]	EPDM	Neoprene	PEEL[d]	PVC	HDPE	LDPE	PP
Type of compound	XL	TP	XL	TP	TP	XL	XL	TP	TP	CX	CX	CX
Matrecon number	44	77	100	55	36	91	90	75	59	105	108	106
Initial thickness, mil	63	30	36	33	23	32	35	7	33	32	31	33
Analytical properties:												
Weight gain, %	21.9	107.2	34.4	31.6	6.7	5.2	41.4	4.5	46.2	0.56	0.51	0.18
Extractables, original, %	11.8	9.1	17.4	4.1	5.5	23.6	21.5	2.7	35.9	...	2.1	...
Extractables, exposed, %	12.3	5.7	15.2	4.0	6.3	22.1	16.9	3.3	26.1	...	2.7	...
Gain in volatiles, %	2.2	42.2	19.7	20.7	2.1	4.3	24.8	4.1	11.9	0	0	0
Change in extractables, %	+0.5	-3.4	-2.2	-0.1	+0.8	-1.5	-4.6	+0.6	-9.8	...	+0.6	...
Physical properties[e]:												
Final thickness, mil	64.3	47.9	40.9	37.9	20.2	36.8	43.5	6.6	36.4	31.5	30.9	32.2
Tensile strength, % retention	107	10	63	48	48	93	49	89	31	88	104	97
Elongation at break, % retention	115	155	79	79	74	106	82	101	89	101	102	105
Stress at 100% elongation, % retention	74	6	46	80	70	81	15	90	28	92	103	110
Tear resistance, % retention	...	14	29	39	79	105	34	90	23	81	86	91
Hardness change, points	-2	-60	-20	-14	-3	-1	-25	-10[f]	-33	-1	-20	-2
Puncture test:												
Stress, % retention	73	20	85	112	101	107	66	77	48	101	92	94
Elongation, % retention	126	127	125	131	142	111	119	100	133	107	108	42

[a] All materials are unreinforced.

[b] A saturated solution of <0.1% TBP in deionized water.

[c] Elasticized polyolefin.

[d] PEEL = Polyester elastomer.

[e] Data for tensile, elongation, S-100, and tear are the averages of measurements made in both machine and transverse directions.

[f] Due to curling of sample, magnitude of change as reported is probably too great.

TABLE 8. PERCENT CHANGE IN WEIGHT FOR LINER SAMPLES IMMERSED IN TRIBUTYL PHOSPHATE (TBP) IN 2 CONCENTRATIONS AT AMBIENT TEMPERATURES

			Weight change, %			
			In 100% TBP		In 0.1% TBP	
Polymer	Liner number	Type of compound	1 day	14 days	500 days	1000 days
Butyl	44	CX	-0.6	-1.6	21.9	23.1
CPE	77	TP	789	a	107.2	120.6
CPE	100	XL	79.1	74.6	34.4	37.5
CSPE	6R	TP	15.0	21.3	30.0	30.1
CSPE	55	TP	13.1	22.9	31.6	31.7
ELPO	36	CX	6.6	15.0	7.3	9.7
EPDM	83R	XL	-5.5	-7.9	6.6	9.8
EPDM	91	XL	-4.5	-14.7	5.2	5.9
HDPE	105	CX	0.1	0.2	0.6	0.5
LDPE	108	CX	0.4	0.9	0.5	0.5
Neoprene	90	XL	98.2	92.6	45.4	41.1
Polyester	75	CX	5.6	7.2	4.6	4.6
PVC	11	TP	1129	a	54.7	52.8
PVC	59	TP	986	a	42.9	40.7
PVC	89	TP	1090	a	49.0	47.5

[a] Too disintegrated to weigh.

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ANALYSIS OF LEACHATE PRODUCTION IN CLOSED HAZARDOUS WASTE LANDFILLS

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ABSTRACT

Hazardous wastes disposed of in landfill sites may continue to drain for several years after site closure. Leachate sources include waste fluids as well as meteoric water trapped in the landfill during construction and operation. Waste fluid may be released via barrel degradation, and interstitial fluid may be released during subsidence and/or compression of waste materials. Water may also continue to enter the landfill through structural faults. Predictions of rates and amounts of leachate production can be developed if the hydraulic parameters and/or specific-yield values for the hazardous waste and backfill materials are known. A literature search was completed, which showed that limited hydraulic parameters and specific-yield information is available. Unit-gradient and specific-yield modeling approaches were evaluated for use at hazardous waste landfills. Specific yield was determined for two data sets: one collected by a commercial hazardous waste site operator and provided to us by the state regulatory agency and the other collected by the authors at a hazardous waste site located in New York State. Specific-yield values of approximately 10 to 16% were calculated for the first data set. These values are within the range of reported values for similarly designed municipal waste landfills. A specific yield of 25% was calculated for recent hourly leachate level data collected by the authors at the New York State site.

INTRODUCTION

Leachate levels rise in closed hazardous waste landfills as waste materials dewater and as meteoric water trapped during construction and operation drains under gravitational influences. This leachate collects on top of the liner and is pumped or drained by gravity via leachate collection systems. Under guidelines developed pursuant to the Resource Conservation and Recovery Act (RCRA), the depth of leachate in the cell above the liner should not exceed 305 millimeters (mm) [1 foot (ft)]. If leachate in the cell lies above the 305-mm standing level, operators will be required to pump or drain the cell until it can be demonstrated that the guideline level has [illegible] achieved. If the cover or liner is no[illegible] functioning properly, additional fluid may be introduced to the landfill. If models can be developed to estimate the fluid drainage rate from a properly functioning closed landfill, deviations from the predicted leachate drainage rate may be used as indicators of cover or liner failure. Few data have been reported on the hydraulic properties of hazardous wastes in closed landfills that would help in the development of these models.

Previous work on leachate production from sanitary landfills has emphasized the concept of field capacity to determine the amount of leachate produced at a landfill (18, 7). Field capacity is defined as the amount of water retained by a porous material after gravity drainage ceases; therefore, in closed landfills where the waste is saturated, field capacity refers to the moisture content after the landfill is drained. Hence, the maximum volume of fluid that may drain from a closed landfill is the difference between the saturated porosity and the field capacity and is denoted as the specific yield or drainable porosity. The specific yield, therefore, is essential in determining the total amount of leachate that may be drained from a landfill.

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PURPOSE

The purpose of this project was to estimate the specific yield of typical hazardous landfilled material. This information can be used in conjunction with leachate level data to estimate the total amount of leachate that may be drained from a landfill.

APPROACH

Our project was divided into three main tasks: 1) to assess the availability of data concerning the hydraulic properties of waste and backfill material, 2) to review the conceptual models applicable to investigating the drainage characteristics of wastes and soil in a landfill environment, and 3) to select an appropriate conceptual model to estimate leachate production from typical hazardous waste sites. With respect to this last task, it should be noted that much of the historical data on leachate production and level used here were collected by hazardous waste site operators and do not necessarily meet standard quality assurance guidelines.

Assessment of Data Availability and Sources

In order to model the flow of water through landfill wastes and soil, several types of data are necessary. To describe the flow of liquid through landfill materials, data must be obtained on the physical packing and disposal techniques practiced in hazardous waste cells. In most landfill scenarios, flow of liquid will occur through saturated and partially saturated media. To represent these types of flow, data concerning the saturated hydraulic conductivity and porosity, and the effects of desaturation on the capillary pressure and hydraulic conductivity are required. In addition, landfilled material may also deform and consolidate. Geotechnical data must be used to determine the effects of loading and consolidation on the saturated void ratio and saturated hydraulic conductivity.

Information on hydraulic properties of wastes and soil drainage are found in the literature from a wide range of disciplines (e.g., chemical, civil, agricultural, and geotechnical engineering; biology; geology; soil physics; and soil chemistry). A comprehensive literature review was undertaken using computer-assisted data bases. The data bases surveyed included NTIS (for government research reports), COMPENDEX (for engineering documents), and BIOSIS (for biological and environmental publications). Key words, such as landfill, drainage, landfill cells, liner, leachate, conductivity, and permeability were used to identify titles and abstracts applicable to this study. In addition, owners and operators of hazardous waste landfills were contacted to determine their disposal techniques and types of waste forms handled at the facility. Results of this literature review are summarized in Table 1.

Conceptual Models

Simplifying the assumptions about the complexity of the flow was necessary in order to model the behavior of hazardous waste landfills. The two major assumptions made for this analysis were that 1) all liquid flow is vertical and 2) the hydraulic properties of the waste materials are uniform throughout the landfill. The rationale and implications of these assumptions are discussed below.

The reduction of the flow geometry from three dimensions to a one-dimensional (1-D) analysis is dictated by the increased complexity and computational costs associated with 2-D and 3-D flow models. Although landfills are 3-D in nature, the assumption of flow only in the vertical direction may be valid for landfills of regular geometry receiving uniform areal recharge. The assumption may not be valid in landfills where surface soils (covers or daily backfill) or surface slopes result in increased runoff in certain areas of the landfill and ponding of precipitation in others. In addition, horizontal hydraulic gradients at the landfill sidewalls or the presence of a

TABLE 1. REPORTED HYDRAULIC AND GEOTECHNICAL PROPERTIES OF HAZARDOUS WASTES

Waste type	Saturated parameters			Partially saturated parameters				Grain size data	Consolidation parameters			Shear strength data	Source
	K_{SAT} (cm/sec)	Porosity	Compacted density (kg/m^3)	K(θ)	θ(H)	θRES	Field Cap.		Cv (cm^2/sec)	E(σ)	% Vol reduction		
Municipal waste		50%					45%						(13)
Municipal waste			340.0				10-14%						(19)
Papermill sludge	1.0×10^{-4} 1.0×10^{-8}												(1)
Flue gas Desulfuritzaion (FGD) sludge	1.0×10^{-4} 1.0×10^{-6}									Yes		Yes	(8)
Solvay soda ash sludge			2.71*					Yes	2.87#	Yes		Yes	(12)
FGD sludge		46-57%	1040-1280					Yes		Yes		Yes	(6)
Municipal waste							20-35%			Yes			(24)
FGD sludge	1.0×10^{-2} 1.0×10^{-5}		2.72-2.52*						0.75-1.05#			Yes	(2)
Lead/zinc mill tailings	3.4×10^{-6} 5.0×10^{-5}	41-57%	2.88-3.02*					Yes					(9)
Uranium mill tailings	2.2×10^{-4}	44%	1.63**										(15)
Municipal waste				Yes	Yes		30-40%						(22)
Uranium mill tailings													(11)
Coarse	6.3×10^{-3}	43.3%	1.48-1.57**	Yes	Yes	7.6%		Yes					
Medium	2.3×10^{-3}	45.8%	1.28-1.50**	Yes	Yes	9.0%		Yes					
Fine	6.7×10^{-7}	66.0%	0.90-1.10**	Yes	Yes	31.0%		Yes					
Fly ash	8.3×10^{-3} 5.0×10^{-2}		0.99-1.50**	Yes Yes	Yes Yes								(2)
Municipal waste							Bulk density versus field capacity data						(7)
Municipal waste			287.0				28.6%						(18)
Spent oil shale										Yes		Yes	(17)
Municipal incinerator residue	$4.1\text{-}6.9 \times 10^{-5}$												(14)
Phosphate tailings	1.2×10^{-4} 2.0×10^{-5}												(23)
Coal mine wastes	1.4×10^{-3} 7.22×10^{-5}												(5)
Waste clays from phosphate mining	1.0×10^{-4} 1.0×10^{-8}								$1.0\text{-}6.0 \times 10^{-4}$				(10)
Fly ash/soil mixture	1.0×10^{-3} 1.5×10^{-6}												(16)
Red mud (aluminum tailings)	$1.2\text{-}20 \times 10^{-7}$								$0.09\text{-}6.9 \times 10^{-2}$				(21)

* - particle density
# - compression index
** - bulk density

shallow water table may produce significant components of horizontal flow.

The second major assumption of uniform hydraulic properties ignores the effects of waste heterogeneity. Typical hazardous waste landfills in the United States handle a wide variety of solid wastes ranging from contaminated soils and sludges in bulk form to drummed waste and polychlorinated biphenol-contaminated transformers. In addition, soil material is often placed between the layers of drums to allow vehicle movement within the landfill cell during operation. Modes of deposition of the wastes range from neat upright or horizontal placement of drums and careful mapping of their location to haphazard disposal of drums and containers by dragline or overhead crane. Bulky waste material is often dumped and spread by bulldozer or front-end loader. In all cases, waste properties cannot be expected to be uniform throughout the landfill.

For this study, however, the focus is not on a complete description of fluid movement throughout the cell, but on that amount of fluid draining to the leachate collection system. The effect of the drainage collection system is to average the local drainage from various portions of the cell. In similar problems of agricultural drainage (4), simplified models of flow mechanisms can be employed because the details are lost in the process of averaging over large areas. As a result of a review of the available research on flow in hazardous waste cells, two types of modeling approaches were chosen for application: unit gradient and specific yield.

Unit Gradient

The unit gradient approach (20) is applicable to soil or bulk waste cells (i.e., those cells receiving a uniform, soil-like waste material). Such cells might be found at private generator/disposer sites where cells are used strictly for a specific kind of waste (e.g., flue-gas desulfurizing sludge from a coal-fired electric utility plant). These waste types would be expected to behave like soils, and drainage from these materials has been analyzed as such (2).

In order to assess the validity of this approach, we compared this unit gradient analysis with a fully implicit 1-D numerical ground-water flow code, UNSAT1D (3). The model was run using 50 equally spaced node points over the 10-m depth of a simulated landfill. The water table was set at the bottom of the fill.

The resulting drainage from the bottom of the simulated waste cell is shown in Figure 1. As can be seen, both solutions exhibit a period of constant drainage (stage one drainage) followed by a rapid decrease in drainage. At later times, the drainage from either solution decreases slowly. The total change of fluid stored in the landfill at the end of 1 year (the length of time for numerical simulation) estimated by the analytical solution was 15% greater than that predicted by the numerical model. Although the analytical method overestimates the length of time required for stage one drainage, the solution closely matches the drainage characteristics at later times, with the drainage rates after 1 year virtually identical between the analytical and numerical solution. Therefore, only the analytical approach need be used in homogeneous bulk waste cells.

Specific Yield

The second approach is for landfill cells that may not contain primarily soil-like material. Currently, many commercial waste sites dispose of drummed, solidified waste. Leachate will be stored in the large voids between the drums as well as in the pore spaces of any backfilled soil. In most cases, the large voids will contain the most leachate. Under drainage conditions, these large voids will easily dewater, leaving the leachate in the backfilled soil behind. The concept of free-draining pore spaces (voids) can be used to model fluid drainage in drum disposal cells.

The specific yield of a porous medium is defined as the volume of drainable pore space per unit volume of porous medium. In a drum disposal cell, the drainable pore space primarily consists of the void spaces between the drums. The volume of the interdrum voids may be estimated using the number of drums in the cell, the amount of backfill soil and bulky waste,

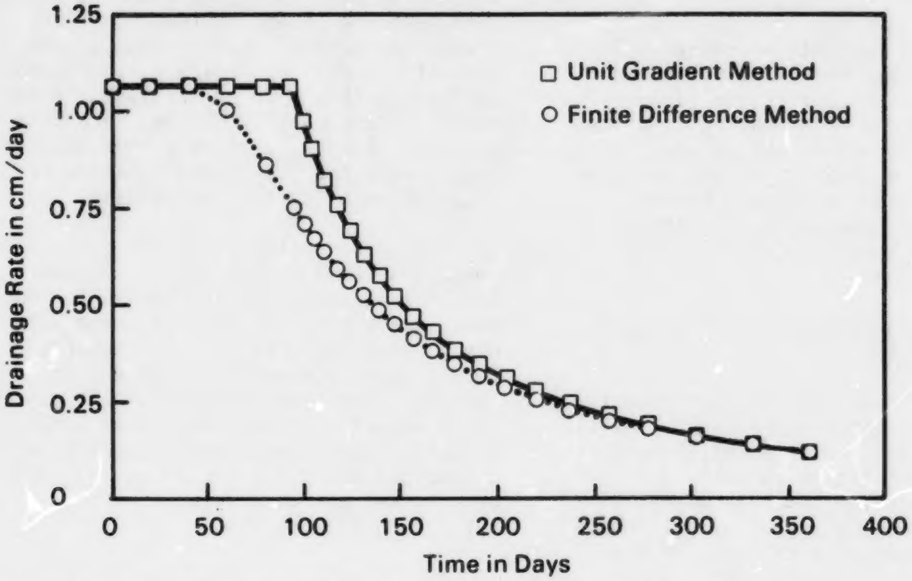

Figure 1. Comparison of drainage rates from Yolo light clay using unit gradient analysis and numerical analysis.

and the overall cell volume. Given an efficient underdrain system (one that will cause little flow blockage), leachate levels in the cell may be predicted using the specific-yield approach.

Using the assumption that the porous material has sufficient time to drain to near equilibrium, the specific yield is given by the following equation:

$$Sy = \frac{Q}{LFV} \times 100 \qquad (1)$$

where Sy is the specific yield, Q is the amount of leachate pumped out of the bottom of the landfill, and LFV is the volume of landfill drained. This Sy value includes the volume of porous and solid materials in the landfill volume. Often the Sy of only the porous material is required and is obtained by dividing the Sy for the entire landfill volume by the ratio of solids to porous, soil-like materials.

PROBLEMS ENCOUNTERED

Limited information on hydraulic properties of bulk wastes (e.g., contaminated soils, sludges), layered materials of different hydraulic properties, and solid materials prevent useful application of the unit gradient approach. The specific-yield values, when properly selected for a given waste material, appear most promising for determination of leachate production.

Leachate production data collected at a commercial hazardous waste landfill required extensive data manipulation to reduce variability in order to develop an estimate of specific yield or drainable porosity of the waste material. The range of specific yields calculated compared favorably with those reported for municipal waste landfills. Nonequilibrium levels in the standpipes affected the analysis in two ways:

1. It was necessary to use the maximum monthly recorded leachate level in each subcell to estimate the actual leachate level in the subcell. Daily variations in the recorded levels were the results of leachate levels not equilibrated because of slow standpipe recharge.

2. Hydraulically connected standpipes often did not respond in a similar manner to leachate pumping. This may be a result of nonequilibrium conditions or measurement error.

RESULTS

The models evaluated represent simplified conceptual models of hazardous waste landfills. Under actual field conditions, no one model may be completely applicable. The models chosen are designed for use under a variety of landfill configurations and waste types. Table 2 outlines the key points of the models and the typical data needed for their application.

Site Description Specific Yield Analysis (Commercial Site)

For this study, we analyzed data from a (<2 acre) hazardous waste landfill site located in a humid environment. The site has many waste cells; each cell is designed to minimize leachate migration into the surrounding environment. The cells are excavated in native clay and lined with both compacted clay and flexible membrane liners. Figure 2 illustrates the general plan of each of the subcells in Cell #3. Data collected for Cell #3 at this site are analyzed in this report. This cell is divided into five subcells; each subcell is hydraulically separated using clay berms to allow the segregation of certain materials.

The data collected by the site operators consisted of daily leachate level measurements in standpipes for each subcell and monthly total leachate volumes pumped from each cell. Two leachate levels were recorded for each day for each subcell: the level before pumping and the level measured immediately after pumping stopped. Level measurements were taken in standpipes using a weighted string, and pumping volumes were measured using totalizing flow meters.

TABLE 2. SUMMARY OF CONCEPTUAL MODELS

Model	Application	Resulting data
Unit gradient or finite difference	Bulk waste forms, known hydraulic properties, uniform initial water content profile, deep landfill	Drainage flux versus time, water content profile versus time
Specific yield	Unknown hydraulic properties, drummed or rapidly draining waste forms	Leachate level versus time

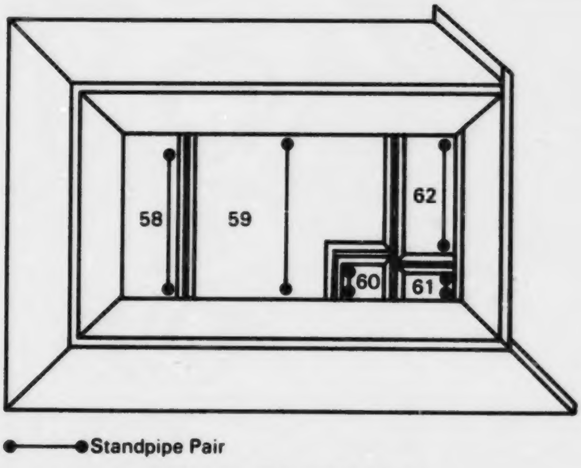

Figure 2. Plan view of Cell #3.

To collect leachate generated during landfilling operation, leachate collection networks are located in each subcell. Each network consists of two perforated concrete standpipes connected to one another by a perforated 0.10-meter (m) clay pipe. The standpipes were constructed as the subcell was filled, so that at no time did the standpipe extend more than 1.21 m above the working floor of the landfill. The bottom liner was sloped toward each standpipe to facilitate drainage. Leachate was pumped from one standpipe in each subcell, and the leachate level was measured daily in each standpipe.

Waste in each of the cells consisted of both bulk (i.e., sludges, contaminated soils) and drummed [i.e., 0.2-m^3 (55-gallon) drums] material. Drums were stacked upright as closely together as possible. Sludge waste material was unloaded into a bermed area built to accept the amount of material coming in on a particular day. Bulk waste material was also placed in each cell using a front-end loader. It was assumed that a volume ratio of two parts bulk waste to one part drummed waste existed in Cell #3 as estimated by landfill operators. At the end of each daily shift, at least 0.15 m of cover material were placed over all exposed drums. This cover material ranged from natural clays and sands to neutralized wastewater sludges. On completion, the cell was sealed with a minimum of 0.91 m of compacted clay, 0.45 m of uncompacted clay or clay/soil mixture, and 0.15 m of top soil, and then seeded with perennial rye and bluegrass.

Cell #3 received waste from the summer of 1979 to the fall of 1982, when a temporary cover was installed to reduce infiltration. Final closure was completed in September 1983. Leachate was removed from the standpipe collection system during cell operation using a portable pump and/or vacuum trucks. Beginning in February 1984, an automatic pumping system was installed that consisted of sump pumps actuated by leachate level.

Results for the Commercial Site

This analysis focuses on interpreting the existing leachate production data from Cell #3 to determine the hydraulic properties of the waste material. To analyze the leachate production from a mixed-waste disposal site, we used field data to estimate the specific yield or drainable porosity. These data are extremely important because they can be used to estimate the total amount of drainable leachate in the landfill. In estimating the specific yield, the leachate level in the landfill is assumed to have reached equilibrium when the monthly maximum value occurs. In cases where the leachate pumping rate is small in comparison to the volume of leachate contained in the landfill, the assumption of instantaneous drainage appears valid. The temporary cap is believed to have significantly reduced fluid input to the cell. Assuming that fluid input to the cell is negligible, the leachate level data taken after September 1982 should have been primarily influenced by the leachate pumping. The data indicate that the volume of leachate pumped declined rapidly after the temporary cover was installed. Table 3 shows the monthly maximum leachate levels in each of the five subcells. These data show clearly decreasing leachate levels in response to leachate pumping. Table 3 also indicates that the leachate levels in all but Subcell #62 decreased at a nearly constant rate, whereas the leachate production rate dropped off in an exponential fashion.

One possible cause for the linear decline in the leachate level was due to the variable geometry of each subcell as it drained. Because the sidewalls of the cell slope, removing a given volume of leachate at high leachate levels will not lower the level as much as when the leachate level is low. If the cell walls were vertical, the leachate level decline would be directly proportional to the leachate production rate. Given the sloping sidewall, however, we expected to see a nonlinear behavior in the leachate level/production ratio. To produce the linear trend in leachate level found in each subcell of Cell #3, a leachate production rate that decreases with time is required (see Table 3). A second possible cause of the trend toward a linear decline in the leachate level may be nonequilibrium conditions in the standpipes. As the leachate level falls, so does the recharge rate to the standpipes because the hydraulic gradient is reduced. In the later stages of dewatering a cell, the leachate levels recorded in standpipes may not represent the actual

TABLE 3. LEACHATE DATA FROM SUBCELLS OF CELL #3 MONTHLY MAXIMUM LEACHATE LEVEL AND VOLUME PUMPED SUBCELL

	Subcells (m)										Total
Date	#58	#58A	#59	#59A	#60	#60A	#61	#61A	#62	#62A	m^3 pumped
Sep 82	7.01	7.07	2.62	2.23	1.43	1.83	1.92	1.89	2.68	6.77	363.40
Oct 82	6.92	6.07	2.87	2.96	1.40	1.43	1.92	1.98	5.49	7.77	155.58
Nov 82	6.49	6.52	1.89	1.68	1.40	1.55	1.92	1.74	7.07	3.14	185.49
Dec 82	...	...	...	...	...	...	...	...	...	...	147.63
Jan 83	6.95	6.71	2.50	2.38	1.43	1.52	1.52	1.58	4.39	7.04	111.81
Feb 83	6.52	6.25	2.87	2.93	1.34	1.25	1.43	1.28	3.78	6.74	109.68
Mar 83	...	...	...	...	...	...	...	...	...	...	90.85
Apr 83	...	...	...	...	...	...	...	...	...	...	90.85
May 83	5.55	4.57	1.68	2.56	1.07	1.52	1.71	1.58	7.35	6.04	72.65
Jun 83	5.73	4.45	1.77	1.49	1.37	0.98	1.40	1.58	3.84	6.64	60.51
Jul 83	5.73	4.57	1.71	1.80	0.85	1.46	1.49	1.40	2.53	5.91	120.52
Aug 83	5.00	4.60	0.79	1.13	2.38	3.81	1.22	1.68	1.95	4.54	81.88
Sep 83	4.51	4.48	0.46	1.07	0.46	0.52	1.10	1.58	1.55	3.51	58.67
Oct 83	4.15	3.29	0.30	0.73	0.34	0.43	0.85	0.98	1.49	2.83	46.61
Nov 83	4.08	3.35	0.49	1.13	0.49	0.30	0.52	0.76	1.04	2.74	38.20
Dec 83	3.99	3.60	1.16	0.67	0.40	0.34	0.61	0.76	2.04	2.47	36.93
Jan 84	3.87	3.54	0.88	0.55	0.43	0.40	0.64	0.67	1.77	2.07	37.01

levels in the landfill but will be lower because the leachate will not flow into the standpipes as quickly.

To test these two possibilities, we calculated the specific yield over two time periods. The first time period was from September 1982 to September 1983, the second from October 1983 to January 1984. The first of these periods reflects the time during which the temporary cover was in place. The second period covers the time the final cover was in place. Cover integrity and collection system efficiency can be determined by examining data from these time periods. If cover leakage occurred from September 1982 to September 1983, the specific yield calculated during this period would be higher than that calculated after permanent closure. This increased specific yield would happen as a result of infiltrating water offsetting the reduction in the leachate level brought about by pumping. Also, the question of equilibrium conditions can be answered. If leachate levels in the standpipe are lower than the actual waste leachate level, then the specific yield calculated during these times would be lower (less fluid drained from a unit decline in head) than that calculated when equilibrium conditions existed.

Using Table 3 data, calculations show that the total saturated cell volume in September 1982 was 47,969 m^3. In September 1983, the saturated cell volume was 22,782 m^3. The total leachate volume pumped from September 1982 to September 1983 was 1,649 m^3. The resulting specific yield is calculated as

$$S_y = \frac{1,649\ m^3}{47,969 - 22,782\ m^3}$$

$$S_y = 0.065 \text{ or } 6.6\%$$

The same method was used to determine the specific yield during the period from October 1983 to January 1984. The total leachate volume pumped during this period was 158 m^3. The specific yield is calculated as

$$S_y = \frac{158\ m^3}{18,340 - 16,893\ m^3}$$

$$S_y = 0.11 \text{ or } 11.0\%$$

The results of these analyses indicate that the specific yield was 6.6% during the first time period and 11.0%

during the second time interval. The increase in specific yield (after the final cover was installed) indicates that the temporary cover was as effective in reducing infiltration as the final cover. Leakage through the temporary cover would have resulted in a higher calculated specific yield because infiltration would tend to offset the effects of leachate pumping. The increase in specific yield during the second time period also indicates that nonequilibrium conditions did not significantly affect the results. In this case, nonequilibrium conditions would have caused leachate levels measured in the standpipes to be lower than those in the landfill, and the specific yield calculated under these conditions should be lower than that calculated from actual leachate levels.

The rise in the specific yield value is most likely the result of the inhomogeneities of the wastes and the subcell variations hidden by averaging the saturated waste volumes.

The specific yields determined in this analysis suggest only an averaged value over the entire landfill. They do, however, represent a range of values for this type of waste. Because approximately two-thirds of Cell #3's volume is taken up by soil or porous waste, the overall specific yield calculated for the landfill is two-thirds of the specific yield of the bulk waste. Using the specific yields calculated for the entire landfill volume of 6.6% and 11% [Equation (1)], the bulk waste specific yields would be 10% and 17% for the two time periods investigated. These figures are in the same range as those reported for municipal waste. Specific yields reported for municipal waste range from 12% to 21% (7, 22). Initial estimates of specific yield or drainable porosity of hazardous waste landfills may be calculated from existing data for municipal waste with a reduction factor accounting for the volume of waste containing no drainable porosity (e.g., drums, transformers) as estimated from waste inventories.

Site Description Specific Yield Analysis (Research Site)

An existing hazardous waste site in New York State was selected for installation of an automated water level recorder* to help determine a typical specific yield value for mixed waste. The automated water level recorder is used to determine the changes in water level associated with leachate removal. This waste site has an ineffective clay cap, which is scheduled for replacement during the summer of 1985. The liner is constructed from two layers of clay separated by gravel with a leak detection system located in the gravel. No detectable leakage from the waste site has occurred. At the end of August 1984, a leachate level of 3.6 m was observed above the top clay liner. The leachate level is being monitored as leachate is removed from the waste site in 25.0 m^3 (6600 gallon) increments.

Results for the Research Site

Figure 3 illustrates the changes in water levels after eight leachate removal events. A specific yield of 25% was calculated for the period August 30 to October 1, 1984. Calculation of specific yield for each pumping event varies greatly. This variation is primarily due to initiation of pumping before measured leachate levels reach equilibrium within the landfill. A calculation of specific yield using the initial leachate level (August 30, 1984) and the maximum leachate level achieved after each of eight sequential pumping events yields the following respective values: 13%, 11%, 19%, 19%, 22%, 24%, 19%, and 25%. The first two lower values may be caused by an overestimation of the soil volume drained, as the engineering drawings indicate the 3.6-m water level is near the upper clay cap layer. Also, the variation may be caused by heterogeneity of the bulk waste. Subsequent measurements in October 1984 indicate specific yields of 23 to 27%. These higher specific yields may indicate additional water input from vertical infiltration through the clay cap. Pumping will cease during winter months, at which time cover and bottom liner permeabilities will be estimated from the change in water level.

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*Product of Handar, Inc., Sunnyvale, CA 94089.

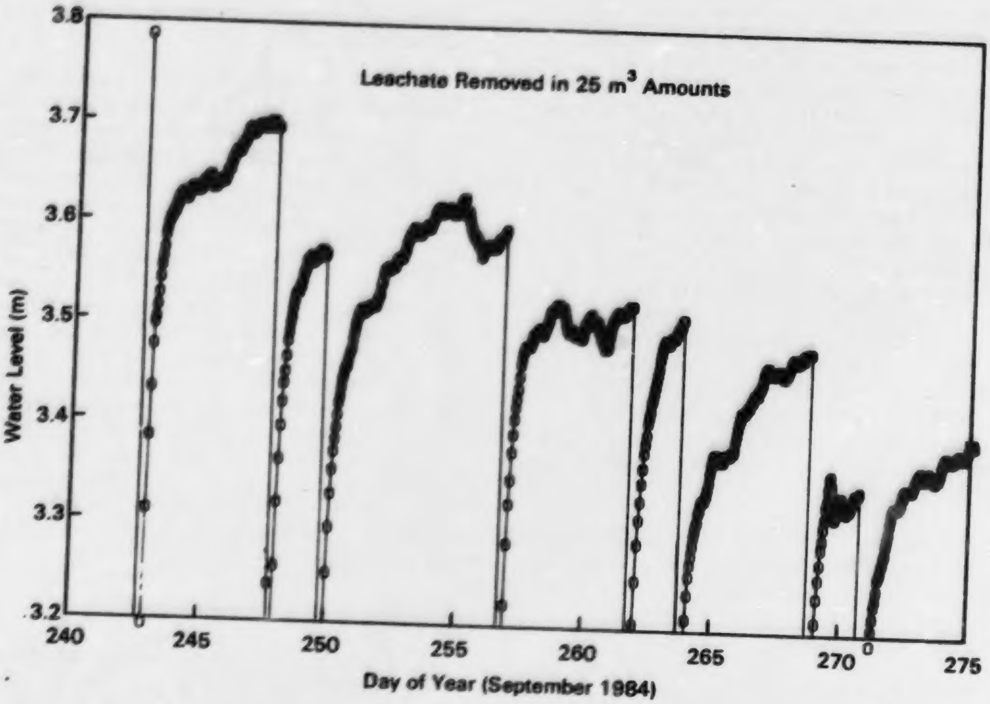

Figure 3. Water level at waste site as a function of pumping.

Concluding Remarks

Based on our results, estimating the total amount of drainable leachate in a cell appears possible by choosing an effective specific yield that accounts for the volume of the waste that does not contribute any drainable pore space (e.g., drums, transformers). Estimates of non-drainable volume can only be obtained from landfill operators and should be required information at all new sites. From the analysis of data collected at a commercial waste landfill, the specific yield of bulk waste ranges from 10 to 17%. Hourly measurements of leachate levels at a research waste landfill in New York State indicated several days may be required for water levels to reach equilibrium. A specific yields of 25% was calculated for an oil-contaminated sandy waste material containing minimal solids.

ACKNOWLEDGEMENTS

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PRECISION AND RELIABILITY OF LABORATORY PERMEABILITY MEASUREMENTS

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ABSTRACT

A preliminary collection of laboratory permeability test data was gathered from nine sources to create a data bank suitable for statistical analysis. The collected data also yielded a survey of the most commonly used permeameters and testing methods for clay liner permeabilities.

This paper describes the methods used to collect and organize the data. Tentative findings are discussed. Consideration is given to the degree of variability found in replicated permeability tests, the question of sample equilibration, parameters of sample preparation which may lead to sizable errors in permeability determinations, and the amount of variability of permeabilities that may be found in location-to-location sampling from within a source of liner material.

INTRODUCTION

Clay liners have been used for many years as barriers below hazardous waste disposal sites. The currently accepted permeability requirement for these liners is 10^{-7} cm/sec (4). The permeability of liner materials is usually determined in geotechnical laboratories under varied conditions and by various testing methods as, for example, by use of fixed wall versus triaxial permeameters. Consequently, there is uncertainty about the precision and reliability of these measurements.

PURPOSE

The objectives of the project were:

a. to collect a limited set of available data on permeability measurements and to describe the most commonly used permeability procedures;

b. to analyze the collected data in order to determine the degree of variability likely to be found among the results of permeability testing, and to identify sources of errors in the testing procedures and estimate their significance;

c. to recommend a statistical protocol for future clay liner permeability tests.

APPROACH

Selection of Data Sources

Permeability data were collected from laboratories that were active either in research or design of clay liners. Nine data sources were identified, based on established contacts, past and on-going research, reputation, and publications. These included four EPA contract sources, two government laboratories, two private consulting firms, and one independent university source.

Collection of Preliminary Data

Data sources were contacted to determine the availability of permeability data. A request was then made to each of the selected sources to provide all suitable permeability data, clay characterization data and test specification data.

As of October 1984, complete data sets have been received from six of the sources; a partial set from one source; and only published reports and theses, but no raw data, from two. Four sets of data have been reduced for computer manipulation, and have been analyzed. Data reduction is in progress for the remaining data sets.

Data Organization and Reduction

Two types of data sheets were used to

organize the technical information.

For each clay for which permeability data had been received, a "Clay Type Sheet" was filled out, containing a description of the clay and information on its origin, batch number, Atterberg limits, classification, texture, sand and silt content, Proctor density and optimum water content, pH, ion exchange capacity, and specific gravity.

For each permeability test for which data had been received, a "Permeability Test Data Sheet" was filled out, containing information on the type of clay used, initial density, water content, void ratio and degree of saturation, type of permeameter used, type of permeability testing apparatus used, types of permeants used and their characteristics, and criteria for terminating the tests.

Attached to each Permeability Test Data Sheet were the corresponding raw permeability test data.

After the data had been organized, they were filed on the computer and were available for analysis.

DISCUSSION OF RESULTS

The following points will be discussed, and examples given below.

a. Although it is somewhat situation-specific, the variability of permeability test results using standard aqueous leachate, standardized compaction, and testing of replicate samples, taken from a uniformly moisterized batch of homogeneous clay soil, is reasonably small relative to other sources of error discussed below.

b. Based on the relatively few permeability tests available, which employ standard leachate over a very large number of pore volumes, there is evidence that equilibrium may be considerably more difficult to establish than is commonly supposed. Furthermore, the error in a permeability determination which may be caused by premature termination may be large relative to the degree of within-batch variability discussed in point a). This implies that the length of testing time typically employed by geotechnical firms may be insufficient.

c. The permeability of a soil sample is extremely sensitive to the parameters of its preparation, such as the molding water content at which it is compacted. Minor variations in these parameters can cause changes in permeability which are large relative to the accuracy of the test itself. Thus, errors introduced during the moisturization step of a permeability determination will not be small relative to the within-batch variability discussed in point a), at least in tests conducted at levels of the parameters where the sensitivity is great. Moreover, in the actual construction of a liner, molding water content and compactive effort cannot be strictly controlled. Under these conditions, it is meaningless to perform a single test at a nominally specified molding water content and standard compactive effort. Instead, the bounds within which molding water content and compactive effort can be guaranteed to be maintained during the construction of the liner must be determined. Then, the proposed material should be tested at perhaps three molding water contents, spanning the range which can be maintained during construction, and conducted at the least level of compactive effort that can be guaranteed during liner application.

d. It is important to assess the degree of uncertainty of permeability determinations due to the nature of the test itself; it is equally important to estimate the amount of variability of permeabilities that might be expected to occur in location-to-location sampling from within a proposed source of liner material, in order to judge the amount of sampling required to ascertain the overall acceptability of the soil within the source. Data from geotechnical firms, which provide information concerning this location-to-location component of variation, indicate that permeabilities of samples taken from various positions within a site exhibit far more variability than do the replicate tests on uniform clays observed in the research setting, even when the samples are relatively homogeneous with respect to their Uniform Soil Classifications, Atterberg limits and grain size information.

Further discussion of these points is divided into three parts: Errors and Variation Associated with Compaction and Testing, Errors and Variation Associated with Moisturization and Molding Water Content, and Location-to-Location Variability.

Errors and Variation Associated with Compaction and Testing

To observe the degree of variation that may be contributed during sample compaction and subsequent testing, samples of a uniform clay soil which are identically prepared and moisturized are required. An example of such data comes from twelve rigid cell tests of Faceville clay conducted by Peirce (5) as part of a comparative study of the effects of chemical leachates. The soil required for the tests was moisturized to a molding water content of 23.7% (optimum moisture content was 23%) in a common batch, then compacted into cells and permeated with water (0.01 N $CaSO_4$) for 321.75 hours, or on average 2.0 pore volumes. Then chemical leachates were introduced into the cells and the tests were continued.

Figure 1 shows the base 10 logarithm of the permeability of each sample over the last 100 hours of water permeation. These permeabilities were averaged over the eleven readings made on each sample during that period. Logs of the permeabilities are plotted rather than raw permeabilities since experience has shown that the degree of variability of log permeabilities is more consistent over different clay types than is the degree of variability of the untransformed permeabilities. This is not surprising, since one would expect the standard deviation of permeabilities to be roughly proportional to their mean, so that in terms of raw permeabilities, less permeable soils would exhibit less variability than more permeable ones. Working with logged data compensates for this effect.

The twelve log permeabilities range from about -7.7 to -7.5 ($2.0\times10^{-8} < k < 3.2\times10^{-8}$) with an average of -7.59 (2.6×10^{-8}) and a standard deviation of 0.061.

Figure 2 shows the behavior of one of the twelve tests, and is typical of the others. After brief instability, the permeabilities became fairly consistent at about 1/2 pore volume of flow. For the particular test shown, permeabilities appear to be increasing slightly during the last 100 hours of the test, but this effect seems quite small. A statistical test to detect systematic time trends across the twelve different tests is in fact significant ($p < 0.01$), but this formal result is practically meaningless, because the tests were run concurrently in the same lab and are jointly subject to minor changes in environmental conditions such as temperature and temporary fluctuations in the main pressure source. Furthermore, the estimated magnitude of the trend was only on the order of a 0.02 increase in log permeability, which could be attributed to such minor causes as a 2°C change in temperature. From a practical point of view, the tests appear to be in equilibrium. Based on the data reviewed to date, had these tests been conducted by a geotechnical firm with the intention of determining the permeability of a soil, they would have been terminated much sooner than the 321.75 hours allowed by Peirce.

The results of Figure 1 give the impression that replicated permeability tests from within a uniformly moisturized batch should not exhibit a great deal of variability, since the range of twelve such replications was only about 0.2 of an order of magnitude. If, however, equilibrium is not achieved, additional error is contributed which may be large in comparison to this variability, as will be seen below.

At the conclusion of 321.75 hours of water permeation, chemical leachates were added. Figure 3 shows the behavior of the tests. Each test is labeled with an identification number (21-28; 35-38). White dots at 271.75 hours represent the final water permeabilities analyzed above. White dots at 1020.9 hours represent the final permeabilities averaged over the last 100 hours of testing (970.9 to 1070.9 hours). Tests 36 and 38 were continued for 2391.33 hours. Black dots represent 100 hour average permeabilities (technically, the points plotted with black dots are medians rather than means, because occasionally an isolated permeability reading in the raw data was extremely high or low, having an undue influence on the average. This did not occur during the final intervals marked by white dots).

The first impression given by Figure 3 is that addition of chemical leachates affected permeability, causing an average increase of about 0.44 orders of magnitude, or 275%. In fact, this is not the case; four of the cells were used as controls with water continued as permeant, and these tests underwent permeability increases similar to the others. This becomes clearer from Figure 4, which shows the increase in log permeability in each test.

Statistical tests to detect an effect of the chemical leachates relative to water, which are based on the differences shown in Figure 4, do not indicate that the chemicals have increased permeability. This is not too surprising, because in this experiment the chemicals were so diluted that many pore volumes would have to be passed before all soil in the cells could be chemically affected.

Possible reasons for the observed increase are:

a. a change in laboratory environment:

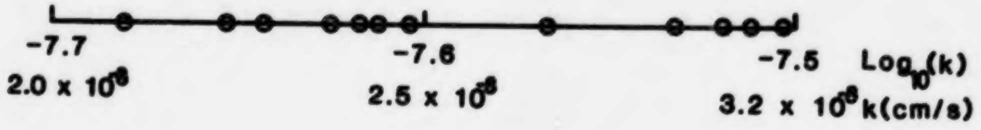

Figure 1. Permeabilities of Faceville Clay after 321.75 hours on water (0.01N $CaSO_4$).

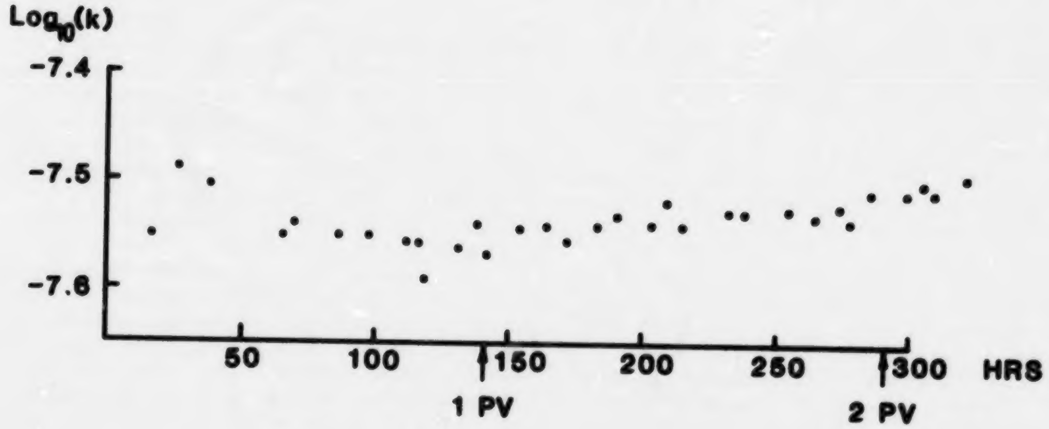

Figure 2. Variation in the permeability of Faceville Clay during 321.75 hours of testing with water (0.01N $CaSO_4$).

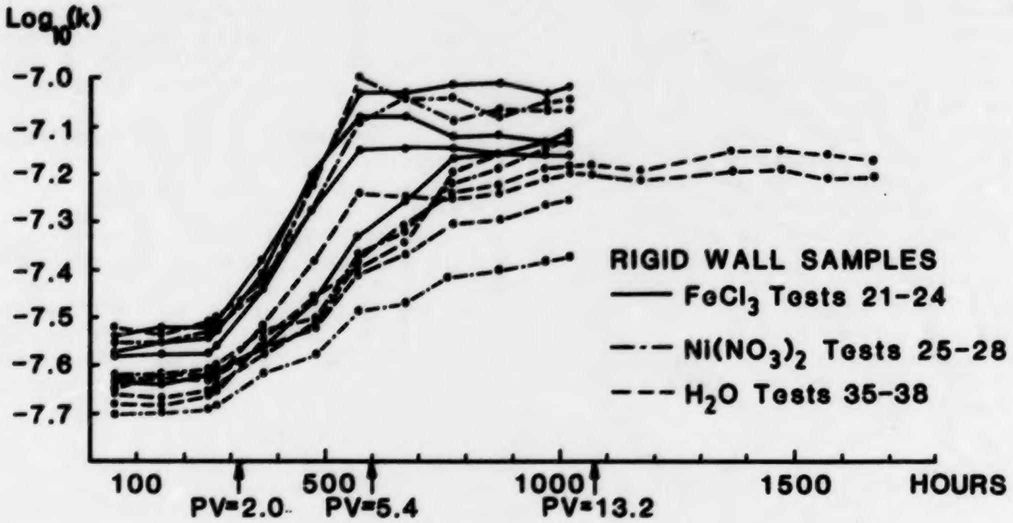

Figure 3. Variation in the permeability of Faceville Clay in rigid mold permeameters.

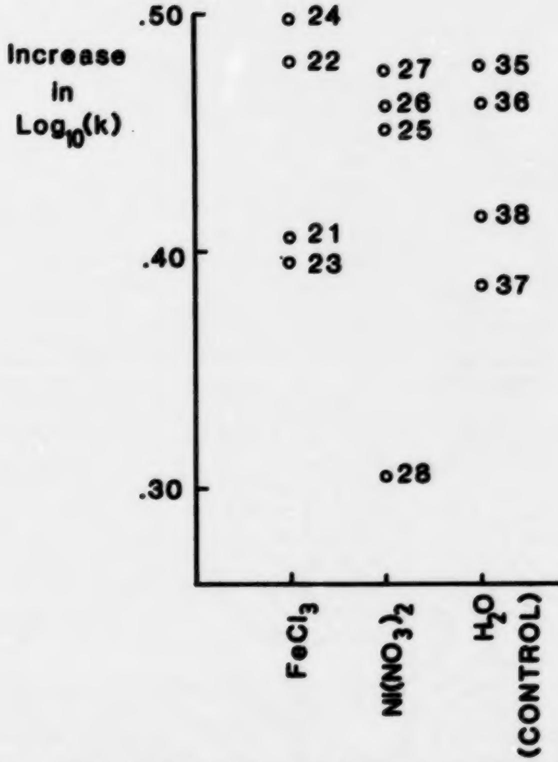

Figure 4. Increase in $Log_{10}(k)$ between 321.75 and 1070.0 hours of flow for Faceville Clay.

This seems extremely unlikely. The effect of temperature fluctuation on viscosity could not have caused an effect of the magnitude observed. Furthermore, pressure gauges were recalibrated each day.

b. sample disturbance caused by depressurization when permeant was switched from water to chemical: This also seems unlikely, because no switchover occurred in the control tests. Further, careful inspection of individual graphs for each test indicate that perhaps the increase began slightly before switchover;

c. slow approach to complete saturation, accompanied by increased permeability: Peirce conducted two triaxial cell tests on Faceville clay in addition to the rigid-wall tests discussed here, one using $FeCl_3$ as a permeant and the other $Ni(NO_3)_2$. Vacuum was applied to these two tests to remove air. Formal statistical comparison between triaxial and rigid-wall results is ambiguous, because the tests were run on separate batches of moisturized clay, were prepared by different technicians and were not run concurrently. In addition, the triaxial specimens were compacted considerably more densely than were the rigid specimens. Nevertheless, an informal comparison is useful. Both triaxial tests start with log permeabilities in the vicinity of -7.1 and decrease to virtually equal equilibria at about -7.29, as shown in Figure 5. There have been few triaxial tests in the data that were reviewed to date, but in most cases, their behavior in coming to equilibrium appears to be better than is the case with rigid cells.

d. time-dependent transition to a more flocculated state, associated with thixotropic hardening: Mitchell, et al., (3) have demonstrated that aging of certain clays prior to permeability testing may lead to increases in permeability consistent with the results seen in this study. Such aging could conceivably occur during the test period.

e. side-wall erosion: This is a distinct possibility in the rigid-wall tests, but it does not explain why their initial permeabilities start well below those of the triaxial tests and equilibrate at roughly the same level.

The failure of rigid-wall tests to equilibrate rapidly has been noted in other data sets as well, including two of four control tests of Brown and Anderson (1) in their comparative study of the effects of organic leachates, and to a lesser degree, in other tests conducted by Peirce. There are two obvious implications:

1. In comparative studies, it is necessary to include control tests in order to provide statistical confirmation of the effect of chemical permeants relative to water.

2. In settings in which the goal is the estimation of the permeability of a soil, the number of pore volumes of flow often seen is inadequate.

In the rigid-wall tests of Faceville clay, the average log permeability increased from -7.59 to -7.15, or over 7 times the initial standard deviation, even though the behavior of the tests was reasonably stable over the first two pore volumes. Even after as many as 10 pore volumes, the equilibrium of several of the tests is questionable. Furthermore, the variability of the determinations at 1070.9 hours is, from Figure 3, somewhat greater than at 321.75 hours, having a standard deviation of .097, although arguably this variability among tests would diminish somewhat as all tests reach equilibrium. This standard deviation gives a measure of the dispersion of permeability determinations which is relevant under the assumption that the introduction of permeants had little or no effect relative to the standard leachate.

To aid in interpretation, a normal probability distribution contains about 95% of its probability within two standard deviations of its mean. Thus, under the assumptions that the permeability determinations are at least roughly normally distributed on the log scale, that the tests are at or near [illegible], and that the sample standard [illegible] of 0.097 provides a reasonably [illegible] estimate of the true standard [illegible] would imply that within-batch [illegible] would generally be within ± 0.2 [illegible] an order of magnitude.

Log permeabilities are moderately correlated with void ratio (.82 correlation at 321.75 hours, .79 correlation at 1070.9 hours; $p < .01$ in both cases) as shown in Figure 6. This indicates that variation in compactive effort may explain a significant portion of the variability observed in the permeabilities of the twelve tests. On the other hand, other causes could lead to differences in void ratio, such as slight inhomogeneity of the soil itself.

Errors and Variation Associated with Moisturization and Molding Water Content

The sources of error discussed in the previous section are associated with the processes of sample compaction and the actual permeability test itself. Additional sources of error which arise during the

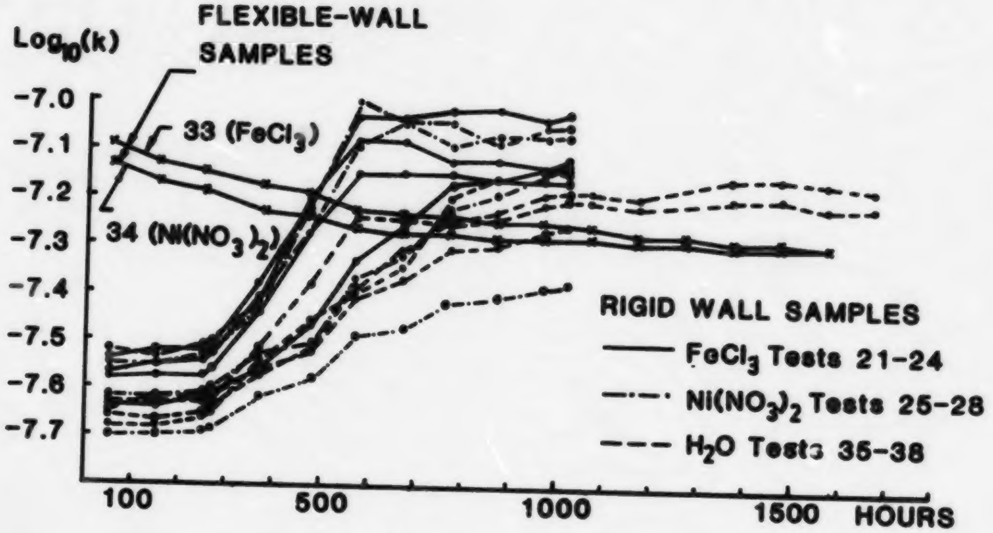

Figure 5. Permeabilities of Faceville Clay – comparison of rigid-wall and flexible-wall samples.

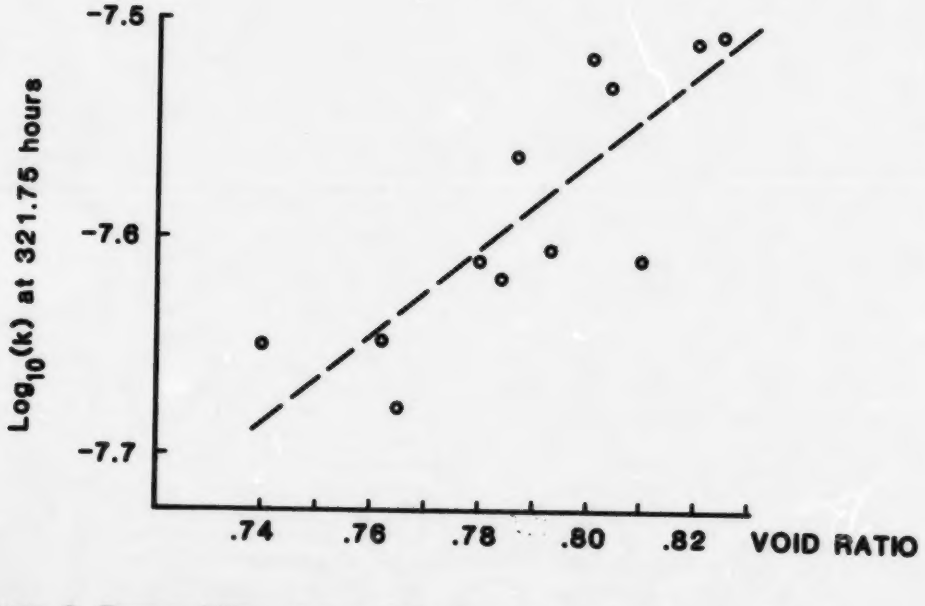

Figure 6. Permeability of Faceville Clay at 321.75 hours versus void ratio.

moisturization of soil do not enter into the variability of the twelve rigid-wall tests of Faceville clay, because the soil required for these tests was moisturized in a common batch.

The molding water content at which a clay soil is compacted may have a profound effect on its permeability. This is illustrated in Figure 7, which shows the sensitivity of the water (0.01 N $CaSO_4$) permeability of Calcareous Smectite to molding water content. These data are due to Brown and Anderson (1). In the vicinity of the optimum water content, the effect is acute. Permeability falls by roughly three orders of magnitude as molding water content increases from 15.9% to 23.5%, or on average about 0.4 of an order of magnitude per 1% water content. A second example, shown in Figure 8, (2) is similar in shape to data for other clays examined by Brown and Anderson. Roberts (6) comments that the curve of Figure 8 is typical.

The steepness of these curves in the vicinity of the optimum water content is directly related to the degree to which a permeability determination may be in error, since the determination of molding water content is itself subject to error. As an example, consider a situation in which it is desired to estimate the permeabilty of the soil in Figure 8 at a molding water content of 19% (roughly 1.5% above optimum) and suppose for the sake of argument that the permeability-molding water content relationship shown in Figure 8 is exact. The slope of the curve at that point is roughly -0.46 orders of magnitude per 1%. It follows that if the actual water content used in the test differs by as little as 0.5% from its nominal value of 19%, an error as great as 0.23 orders of magnitude (or since $10^{.23} = 1.70$, an error of 70%) would be incurred in the permeability estimate, even if the permeability test itself were perfectly accurate. While an error of this magnitude may in certain circumstances be considered to be acceptable, it is not negligible relative to the variability of permeability measurements made on samples compacted from a common batch of moisturized clay. Thus, using the Faceville clay tested by Peirce as an example, it is seen from Figure 3 that all twelve final log permeabilities lie within the interval (-7.0,-7.4), whose half-width is less than the error incurred by a 0.5% error in molding water content in the hypothetical example.

This example shows that (except for tests conducted near that molding water content required to minimize permeability) errors in water content determination may play a significant role in explaining variability of permeability measurements obtained on independently moisturized specimens.

Location-to-Location Variability

The sources of error and variation discussed above relate to inconsistency of sample preparation and test methods as applied to essentially identical soil samples. A separate component of variance which warrants consideration is due, not to errors in testing, but rather to inhomogeneity of soils to the degree which may be expected to occur within a potential source of liner material.

The analysis to be described in this Section was conducted to obtain a rough estimate of the total variability of permeability tests performed on samples selected from various locations within a proposed source of liner material. Such an estimate is needed, for example, to provide judgment on the amount of sampling required in order to obtain sufficiently precise estimates of such characteristics as the median permeability throughout the proposed source or to perform statistical hypothesis tests concerning its suitability for use as a liner material.

The amount of variability found among such test results should depend rather strongly upon the uniformity of the soil source in question. Thus, in this analysis we considered the variability of permeability determinations taken from within soil sources which were geographically contiguous, each of which was confined to a single Unified Soil Classification, with Atterberg limits and grain size data restricted to relatively narrow bounds. Analysis of variance (8) was used to estimate the amount of variability within each source.

Data were provided by an independent soil testing firm. They consist of 23 permeability tests of samples collected from four different sites, all of which employed rigid-wall permeameters and constant head methods. Water was used exclusively as the permeant. There was no replication of testing on soils from the same location. That is, all samples were taken from different bag samples within the four sites.

The number of pore volumes passed during the tests varied from less than .05 pore volume to more than one pore volume,

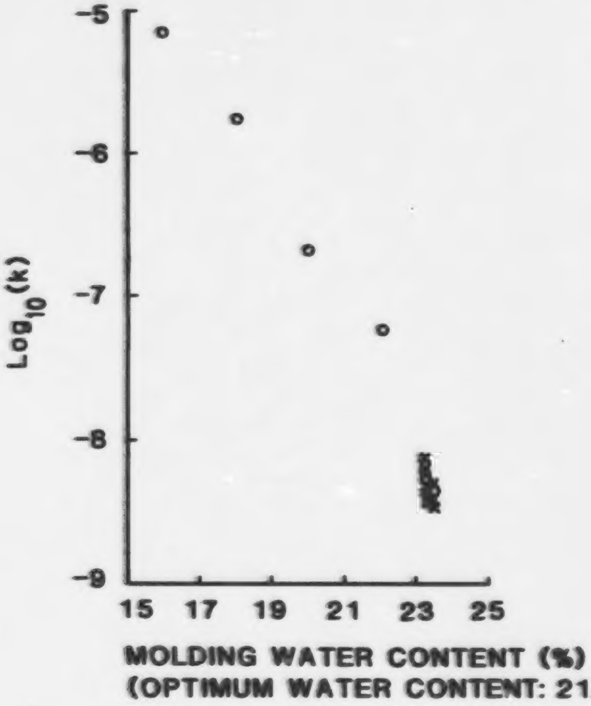

Figure 7. Permeability of calcareous smectite versus molding water content.

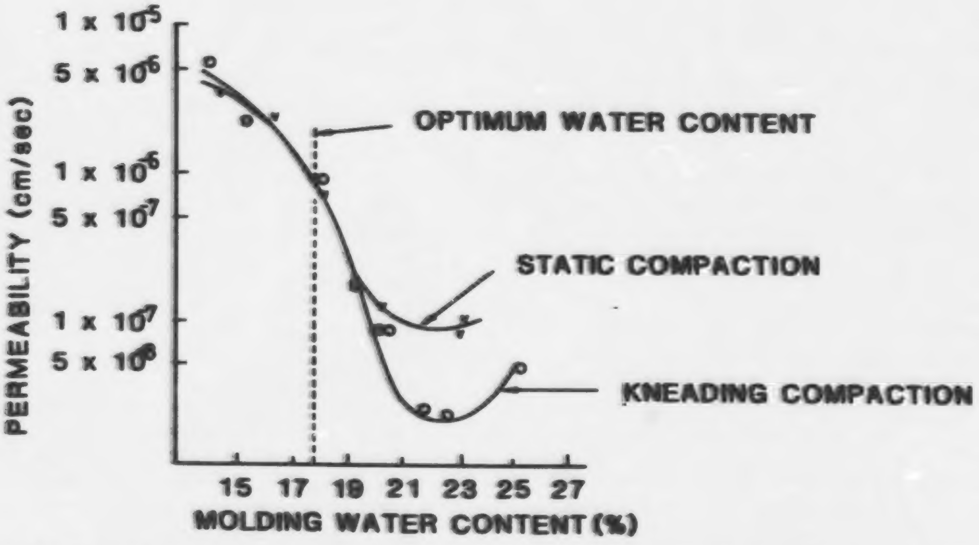

Figure 8. Permeability versus molding water content (Mitchell, 1976).

TABLE I

SITE	UNIFIED SOIL CLASSIFICATION	NUMBER OF SPECIMENS IN GROUP	RANGE IN LIQUID LIMIT	RANGE IN PLASTICITY INDEX	RANGE IN PERCENT FINES	AVERAGE LOG_{10} PERMEABILITY
S1	CL	5	25-33	8-12	67-90	-5.97
S2	CL	2	30-32	15-17	64-72	-8.44
S2	CL-CM	6	18-20	5-6	52-58	-7.49
S3	CL	2	35-41	18-23	80-80	-8.14
S3	CL-CM	2	18-20	4-6	50-51	-7.64
S3	SM-SC	2	19-20	5-6	37-49	-6.55
S4	CL	4	24-33	10-17	65-68	-7.82
		23				

with perhaps one-quarter pore volume being typical.

Within the four sites, samples may be grouped according to their Unified Soil Classification as shown in Table I.

The permeability of each specimen was computed based on its total flow. Then, the within-group variance of log permeabilities, pooled over the seven soil groups of Table I, was computed to provide a rough numerical measure of the degree of variability that may be expected in sampling at different locations from within soils of uniformity comparable to those summarized in Table I.

The within-group variance was 0.22, corresponding to a standard deviation of $\sqrt{.22} = 0.47$. For purposes of interpretation of this result, it may be compared to the standard deviation of 0.097 exhibited by the log permeabilities of Peirce's rigid-wall tests on Faceville clay over the last 100 hours of his tests. Thus on a logarithmic scale, the dispersion of the permeabilities from location-to-location within the soil groups is over four times as great as that exhibited by the final permeabilities graphed in Figure 3.

Using analysis of covariance (8) it is possible to correlate a portion of the variation of permeabilities within the soil groups to factors having to do with variation of the parameters under which the tests were carried out (e.g., molding water content and void ratio) although sufficient data are not available to do this in a very rigorous fashion. After adjustment, the within-group standard deviation is 0.39.

It may be argued that these tests are not comparable to Peirce's in that the volumes of permeant passed were not sufficient to establish equilibrium. Nevertheless, these data indicate that considerable variability in permeabilities is to be expected in location-to-location sampling, even within soil sources which are reasonably uniform with respect to Atterberg limits and grain size information. Further, the point is made that the level of variability that might be observed in data generated in the research environment, with uniform test clays, may have little relationship to that observed in situations where location-to-location variability of permeability within a proposed source of liner material need be considered.

TENTATIVE CONCLUSIONS

Even when standard leachate (0.01N $CaSO_4$) is used as permeant liquid, the establishment of equilibrium may require several pore volumes of flow. In Peirce's Faceville series, permeabilities nearly tripled after having appeared to be nearly stable over the first two pore volumes. Rigid permeameters appear to be less inclined to rapid equilibration than do triaxial permeameters.

The variability of properly equilibrated permeability tests, which are replicated under uniform conditions, appears to be reasonably small in cases where standard leachate is used as permeant liquid. The amount of between-test variability exhibited in the Faceville series is reasonably representative of tests which are replicated within uniformly moisturized batches of homogeneous clay soil. Replication on independently moisturized batches involves an additional component of variance, whose magnitude depends strongly on the degree to which molding water content varies, the slope of the permeability-water content curve in the neighborhood within which tests are conducted, and the homogeneity of the soil from batch-to-batch.

For permeants other than standard, the variability of test results appears to be quite specific to the particular clay-chemical combination under investigation.

The permeability of a specimen is very sensitive to the water content at which it is molded. Unless a test is conducted at a density and moisture content which closely approximates actual field conditions, the results of the test may completely misrepresent future performance of the liner material.

Location-to-location variability of the permeabilities of samples collected from within a source of liner material may be considerable, even when such samples appear to be similar in terms of Atterberg limits and grain size information.

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SIMULATING LANDFILL COVER SUBSIDENCE

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ABSTRACT

The subsidence response of a landfill cover system's clay layer was modeled at reduced scale using a geotechnical centrifuge. The prototype structures for the clay model layers were the multi-layered covers presently being tested as part of an EPA-sponsored investigation of hydrologic isolation. These structures consisted of three 2 ft (0.61 m) layers, with a compacted clay layer at the bottom, then sand, and finally top soil. The clay layer was modeled in the laboratory at 1/48, 1/32, 1/24, and 1/16 scales, and was surcharged with lead shot to simulate the overburden due to the other two layers. The scope of the work was limited to clay layers subjected to "slump", a type of subsidence characterized by localized void spaces beneath the covers. Data were collected to investigate the effects of cavity width and clay moisture content. Qualitative descriptions of the failure mechanisms were developed.

Clay layer behavior was very dependent upon both the moisture content and cavity width. Field cavity widths of 4 ft (1.2 m) were successfully bridged by all models tested at that width, but failures and extensive cracking occurred for many models simulating both 8 ft (2.4 m) and 12 ft (3.7 m) field cavities. Collapse of the samples at the 8 ft width occurred at optimum moisture content (which also was the plastic limit of the clay soil), while sample collapse at the 12 ft width occurred at moisture contents 3 percent below optimum. Qualitatively, soil cracking features were similar to those observed and reported at field sites investigated by other workers.

The centrifuge modeling procedure has promise as an investigative tool to determine the physical mechanisms of subsidence and appears capable of testing a wide variety of different configurations for subsidence response in order to determine optimum landfill cover designs.

INTRODUCTION

The most common and often the most damaging failure mechanism that has been observed in low-level radioactive waste landfills is the inability of cover systems to adjust to short- and long-term subsidence of the underlying waste and backfill materials (5). Volume reduction and subsidence of the trench fill can result from the following processes:

- collapse of structural bridges formed by random dumping of waste containers.
- collapse of voids in the backfill soil that remain after poor soil compaction.
- filtering of backfill soil into unfilled void spaces between waste containers.
- rupturing, degradation and collapse of waste containers.
- percolation of liquid wastes from ruptured containers.

- dissolution of solid wastes in ruptured containers.
- chemical degradation and consolidation of waste materials.
- pumping of leachate from disposal trenches.
- consolidation of backfill soil.

Figure 1 illustrates the processes leading to localized cover subsidence (potholes and slumping). The process of particular interest in this study is slumping, steps (d) and (e) of Figure 1, in which the trench cover bridges a void space. Collapse of the cover will result when the void space reaches sufficient size. Subsidence can result in a breach of cover integrity and can jeopardize functional performance of the cover system. The most damaging effects of subsidence are significantly increased infiltration rates due to ponding and crack formation, direct exposure of the waste cell to precipitation and surface runoff, release of contaminants, and accelerated bio-degradation of the buried wastes (5).

Concern has been expressed that eventual subsidence will compromise the hydrologic integrity of landfills containing hazardous waste components, with subsequent leaching of toxics into groundwater regimes. Since some hazardous waste landfills are known to contain materials in bulk form, in metal or paper drums, and possibly in other decomposable containers; and because it is known that the interstices among these containers are not always completely filled with packed earth, subsidence may occur after some time has elapsed and the landfilled material settles and consolidates. The cover system over a completed landfill forms the barrier to keep out infiltrating precipitation. Breaches in this cover may occur if subsidence of the underlying material occurs, and causes the cover system to change in structure from its original construction. The degree of subsidence and the corresponding amount of cover damage and detriment to the impervious cover system greatly depends on the structural competence of the underlying landfilled material over time. This latter factor has not yet been very well described for modern hazardous waste landfill structures.

PURPOSE

The main force which causes cover slumping is the soil cover self-weight. Centrifuge model testing was selected for this study because it is the only laboratory procedure which can reproduce the effects of gravity in a reduced-scale model (1). The use of centrifuge soil modeling is becoming an accepted method for verifying laboratory soil parameters, geotechnical design assumptions and safety factors (6). It has also been applied to complex problems of soil response, for which acceptable solutions have yet to be developed (1).

The basic concept of centrifuge modeling is to create a scale model that is similar in geometry, material properties, and boundary conditions, to the full-scale prototype, and to subject the scale model to a radial acceleration via centrifugation such that the increase in self-weight stresses in the model matches those at corresponding points in the prototype. If the model dimensions are scaled by a factor $1/N$ as compared to the prototype dimensions, then an acceleration field of N times the acceleration of gravity is required for stress similarity between model and prototype (2).

Centrifuge testing can also satisfy geometric, material, boundary loading, strain, transient fluid flow, and other similarity conditions in a small-scale physical model. These conditions must be verified by experimental evidence, which may be from prototype physical tests, analytical or computer models, or centrifuge model tests at various scales (2).

APPROACH

The behavior of soil structures is influenced by many complex factors such as:

- solid-liquid-gas phase relationships and interactions.
- particle mineralogy, chemistry, size, grading, shape, and structure.
- pore liquid and gas composition, distribution, and pressure.
- previous stress history.
- boundary conditions.
- construction practices.

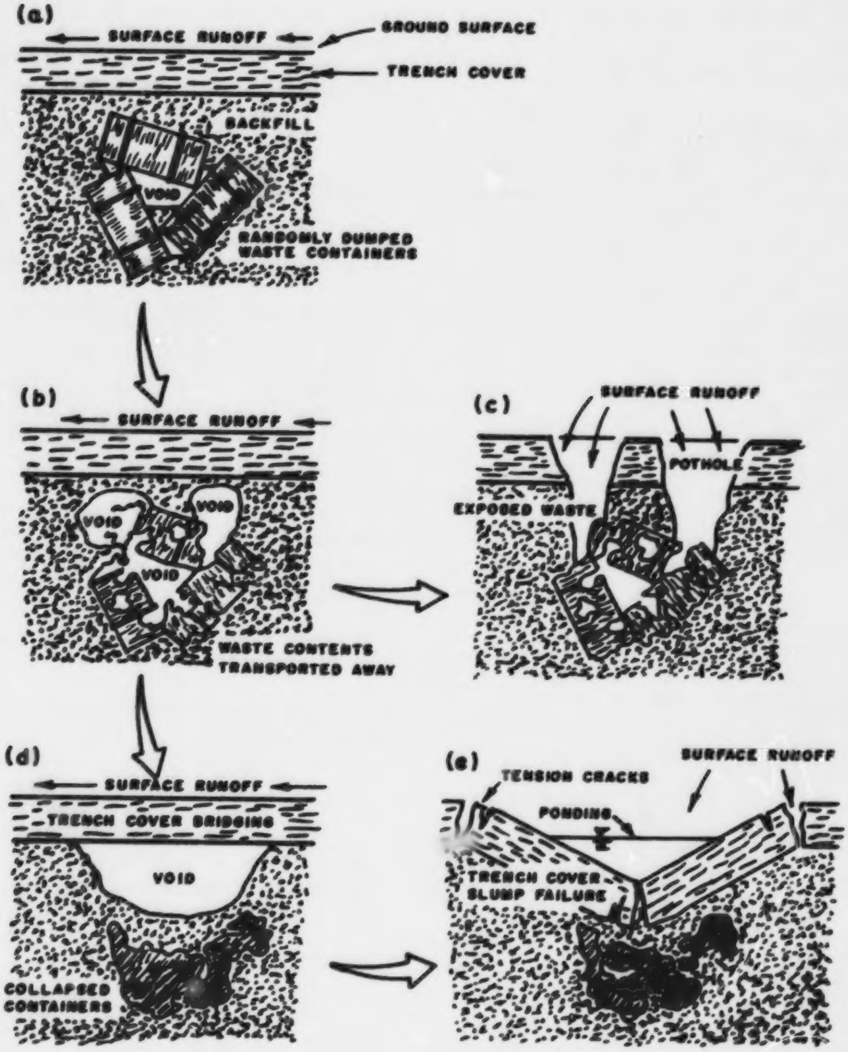
(a)
SURFACE RUNOFF
GROUND SURFACE
TRENCH COVER
BACKFILL
VOID
RANDOMLY DUMPED WASTE CONTAINERS
(b)
SURFACE RUNOFF
VOID
VOID
VOID
WASTE CONTENTS TRANSPORTED AWAY
(c)
SURFACE RUNOFF
POTHOLE
EXPOSED WASTE
(d)
SURFACE RUNOFF
TRENCH COVER BRIDGING
VOID
COLLAPSED CONTAINERS
(e)
TENSION CRACKS
SURFACE RUNOFF
PONDING
TRENCH COVER SLUMP FAILURE

Figure 1. Processes leading to localized cover subsidence (potholes and slump failures.)

(a) Container bridging and poor compaction leave void spaces in waste zone.
(b) Containers degrade, waste leaches out, and soil filters into containers.
Void spaces enlarge, coalesce and migrate upward.
(c) Piping of erodable soils results in pothole formation.
(d) Cohesion of trench cover soil allows bridging over expanding voids.
(e) Collapse of the trench cover into the void results in slump failure.

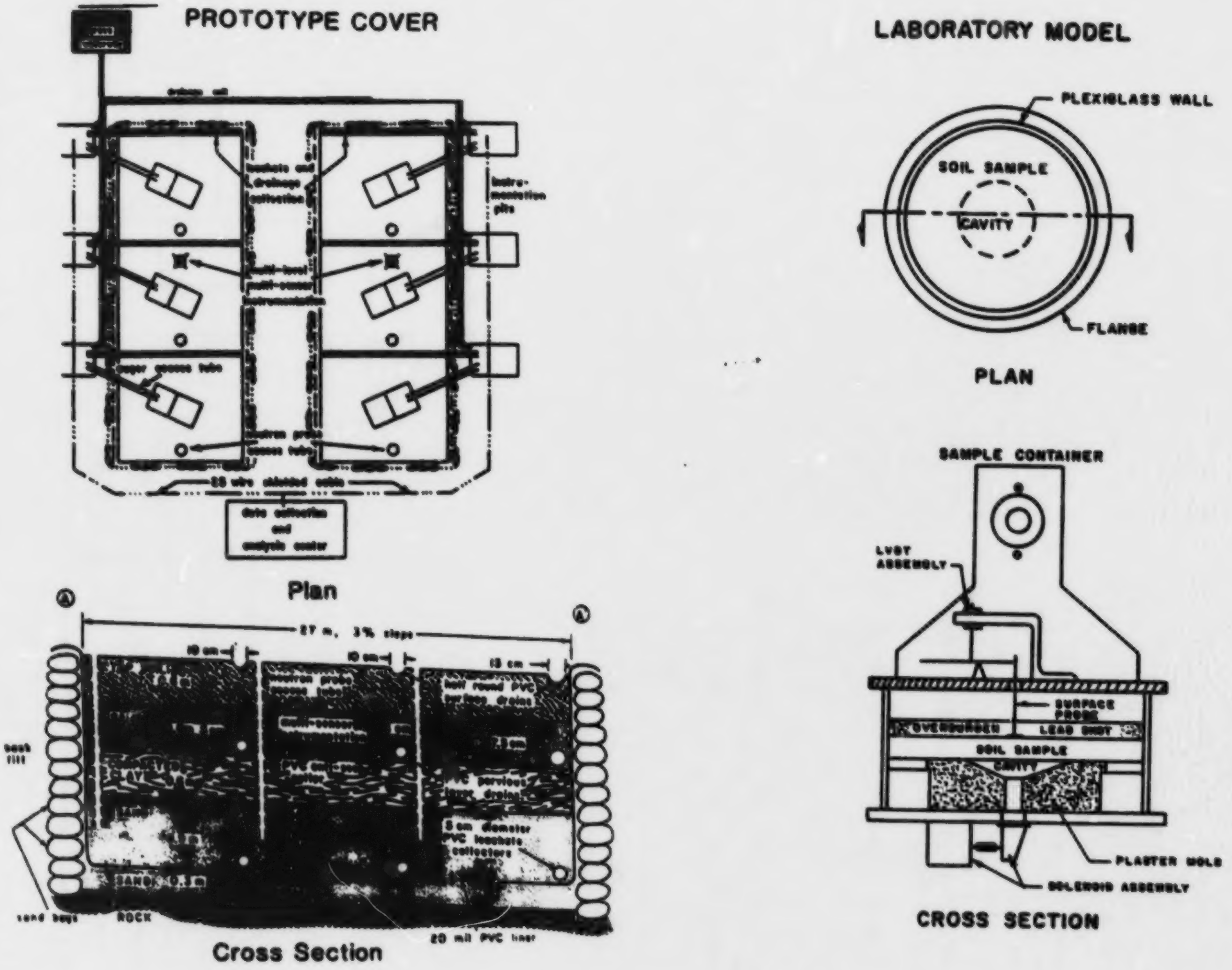
PROTOTYPE COVER
leachate and drainage collection
instrumentation pits
multi-level multi-sensor instrumentation
auger access tube
neutron probe access tube
25 wire shielded cable
data collection and analysis center
Plan
A
27 m, 3% slope
10 cm
10 cm
13 cm
neutron probe access tube
half round PVC surface drains
multi-sensor
2.3 m
PVC pervious layer drains
5 cm diameter PVC leachate collectors
SAND 0.3 m
back fill
sand bags
ROCK
20 mil PVC liner
Cross Section
LABORATORY MODEL
PLEXIGLASS WALL
SOIL SAMPLE
CAVITY
FLANGE
PLAN
SAMPLE CONTAINER
LVDT ASSEMBLY
SURFACE PROBE
OVERBURDEN
LEAD SHOT
SOIL SAMPLE
CAVITY
PLASTER MOLD
SOLENOID ASSEMBLY
CROSS SECTION

Figure 2. Plan and cross section of the field (prototype) covers and the laboratory (centrifuge) models.

The unknown levels of uncertainty and variability associated with these and other factors can make it difficult, and in some cases impossible, to use either theoretical or empirical methods for predicting field soil behavior with an acceptable degree of confidence. As a result, it is often necessary to test geotechnical designs to verify that they will function as intended. The most reliable verification method is to construct full-scale structures for testing. However, the use of full-scale testing is limited due to cost, time, and material handling considerations. Due to these limitations, scale model testing is often used for predicting unknown modes of soil behavior and for verifying geotechnical design concepts (6).

Side-by-side comparisons of the prototype (field) covers and the laboratory (centrifuge) models are shown in Figure 2. The auger access tubes will allow removal of sand from beneath the field cover for future full-scale subsidence testing. The plots were instrumented to monitor soil moisture using tensiometers, resistance (gypsum) blocks, thermocouples, tipping buckets and a weather station. Hydrological isolation of the plots from the surrounding ground is provided by a PVC liner underlying the whole cover system. The field covers are more fully described by Warner et al. (7).

The centrifuge used for this study was a horizontal rotating arm mounted on a vertical drive shaft powered by an electric motor. The centrifuge consisted of six basic sub-units, which were 1) sample container and counterweight, 2) rotating arm, 3) power supply, transfer and control system, 4) structural frame, 5) sand bag wall and protective housing, and 6) data collection and slip-ring assembly.

Figure 3 shows a cross-section of the centrifuge. The 0.61 m (2 ft) thick prototype clay layer was modeled in the centrifuge using either a 1.27 cm (0.5 in.), 1.90 cm (0.75 in.), 2.54 cm (1.0 in.), or 3.81 cm (1.5 in.) thick soil sample accelerated to 48, 32, 24, or 16 g's, respectively. The inner diameter of the cylindrical container in which the soil sample was placed was 29.2 cm (11.5 in.). Plaster molds were used to simulate cavities beneath the soil samples. Several were constructed to be equivalent to 1.2 m (4 ft), 2.4 m (8 ft), and 3.7 m (12 ft) diameter cavities in the field. Lead shot was used above the clay layer to simulate a top soil overburden. Lead shot was also used to initially fill the cavity beneath the layer. A solenoid valve at the base of the sample container was activated during the test to release this shot and thus form the cavity. The models were circular for radial symmetry.

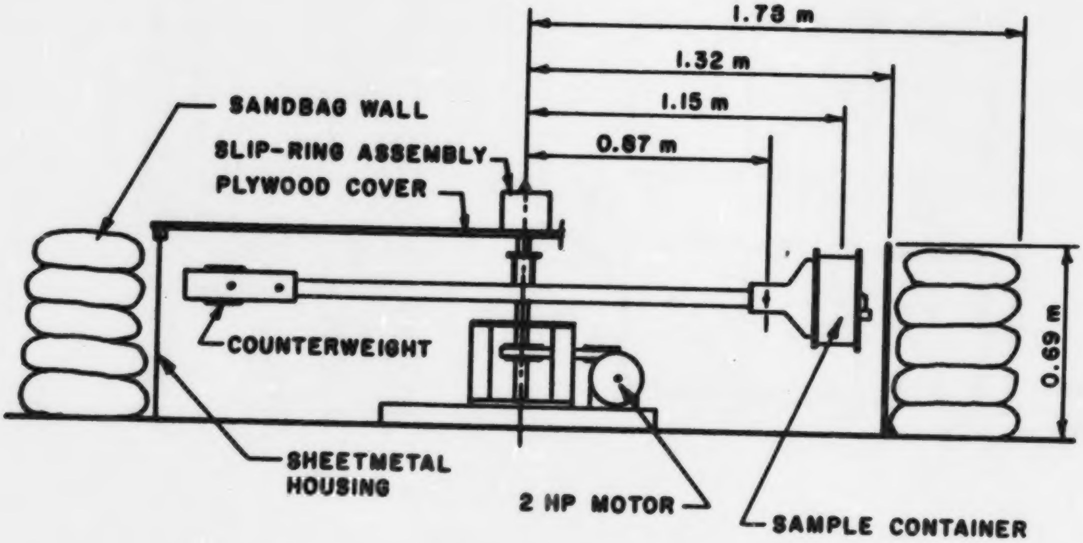

Figure 3. Cross section of centrifuge.

Surface elevation measurements were made with a dial-gauge. A template was temporarily attached to the test capsule, and used to locate the points to be measured. It was made from a 38 cm (15 in.) square sheet of plexiglas, with dial-gauge guide holes drilled on 1.27 cm (0.5 in.) centers in a 23 x 23 grid pattern. Each hole was identified by a set of reference coordinates.

During each test, subsidence was recorded by monitoring the mid-point deflection of the soil layer using a probe. A linear voltage displacement transducer (LVDT) mounted on the top plate of the sample container was used to convert the movement of a surface displacement probe into a voltage that was linked to a strip chart recorder via the slip ring assembly. Physical connection between the LVDT and the probe was made with a balanced lever, which minimized the additional load applied to the soil surface. The elapsed time and the LVDT output, as read from a voltmeter, were written down on the chart record at regular time intervals during the test and when rapid changes in deformation were observed.

The data recorded for each test are listed below:

- model scale factor
- centrifuge speed
- radius to the sample container's centroid
- cavity geometry
- number of blows for clay layer compaction
- clay layer weight
- clay layer moisture content values
- lead shot surcharge weight
- initial and subsided elevations of points on the surface of the clay layer
- time allowed for pore pressure dissipation
- midpoint subsidence of the clay layer
- photographs of the subsided clay layer
- qualitative description of the test

These parameters were selected based on published centrifugal modeling studies (1, 6).

PROBLEMS ENCOUNTERED

Construction of the experimental apparatus and development of the model testing procedure resulted in a number of problems. The two most significant problems are discussed.

The first problem was finding a laboratory testing technique suitable for simulating subsidence. There are generally two approaches to soil model testing which are commonly used to fulfill similarity requirements between a prototype and a model. They include:

1) Build a model at a reduced scale using the same material as in the prototype and subject it to the same forces experienced by the prototype, or

2) Build a model at a reduced scale using a substitute material that is proportionally weaker than the prototype and subject it to forces that are proportionally weaker than those experienced by the prototype.

In general, it is easier to reproduce the forces that act on one body in another body than to reproduce soil behavior in a different material. The second technique has been applied when a known strength parameter can be assumed to govern soil behavior. For example, if a soil can be considered to behave elastically, a model could be built from material having a proportionally lower elastic modulus than the prototype material. The forces applied to the model should be reduced accordingly (4). While geometric, boundary loading and stress similarity can be readily approached with this technique, the actual soil behavior is often too uncertain and/or too complex for material similarity to be fulfilled. As a result, the first technique is most commonly used to model soil behavior (1), and was the reason for the selection of the centrifuge technique.

The second major problem was interpreting the experimental results for prediction of the prototype cover system's subsidence behavior. A common procedure used in geotechnical centrifuge modeling to determine the relationship between different variables and the behavior of the soil sample is the "Modeling of Models" method. With this method, models with different physical dimensions are accelerated so that they are dimensionally consistent. Variability in the behavior of the samples can then be observed and used to better predict the behavior of the prototype structure. This second problem was addressed as follows.

Four clay-layer models were constructed and tested in the centrifuge with different thicknesses, but were otherwise dimensionally consistent. All four had moisture contents of approximately 27 percent, equal compaction intensity, and were bridging a 2.4 m (8 ft) equivalent field-size cavity. The model thicknesses were 1.3 cm (0.5 in.), 1.9 cm (0.75 in.), 2.5 cm (1 in.), and 3.8 cm (1.5 in.), with corresponding accelerations of 48, 32, 24, and 16 g's. Thus, they all simulated the 0.6 m (2 ft) field thickness. All four of these samples bridged the cavity without failure, but a small vertical displacement was measured in each after reaching equilbrium. The relationship between this position and the scaling factor appeared to be inversely proportional to the cube of the scaling factor. Data from these four tests seem to indicate that the displacement in the field prototype would be much greater than the comparable displacement in the laboratory models, thereby leading to the conclusion that the prototype cover would fail sooner or at lower moisture contents than the models. Because of the limited number of tests, this conclusion is tentative, and conclusive verification through more extensive testing is necessary.

RESULTS

Soil Properties

Model tests were carried out using a silty, inorganic clay (CH). Table 1 lists some properties of this material, which was from the stockpile used for the field cover plot study.

TABLE 1. SOIL PROPERTIES

Optimum moisture content (standard Proctor)	25 %
Maximum dry density (standard Proctor)	98 pcf
Percent sand	25 %
Percent silt	37 %
Percent clay	38 %
Specific gravity	2.82
Liquid limit	57 %
Plastic limit	26 %
Plasticity index	31 %

Behavior of Model Clay Layers

Figure 4 illustrates the typical output recorded by the strip chart recorder during the different phases of a test. Following startup of the centrifuge, the sample was accelerated for a time period to allow pore pressure dissipation within the sample. Then the cavity was created by activating the solenoid valve and releasing the lead shot. Three types of responses then resulted. Some samples failed immediately (Type 1), others reached an equilibrium displacement which did not change over time (Type 3), and some samples had a rapid initial displacement, which subsequently slowed down before the final collapse (Type 2).

The following observations of sample behavior were noted.

- Small displacements were recorded for every test as the centrifuge was brought up to speed. These initial drops were attributed to the re-seating of the movable components in the sample container with increased acceleration.

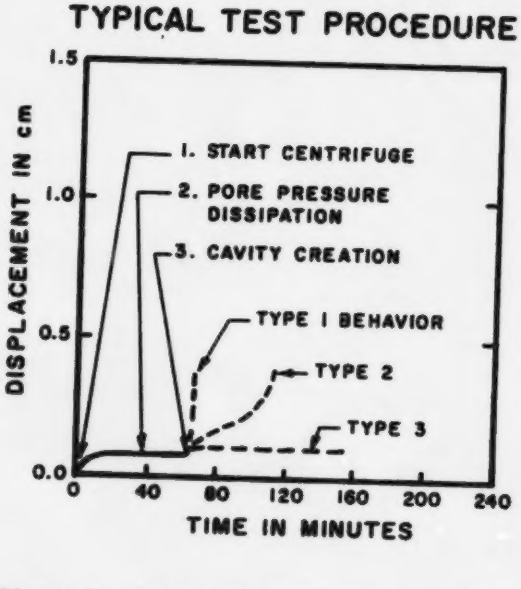

Figure 4. Output of strip-chart recorder during centrifuge test run showing all three types of soil behavior.

- During the pore pressure dissipation period, rapidly decreasing rates of displacement were observed during all tests. This behavior was probably caused by consolidation of the clay layer.
- Measurable subsidence of all clay layers was recorded at the instant of cavity formation. For dry soils and/or small cavities, the magnitude of subsidence was small. No apparent changes in the clay layers were visible. For wet soil and/or large cavities, immediate collapse resulted.

The majority of tests were conducted using one of three soil thickness configurations. For each configuration, tests were conducted for a number of different soil moisture contents. Figures 5, 6, and 7 show that increasing moisture content greatly decreased the ability of the clay layer to withstand subsidence, and Figures 5 and 6 clearly demonstrate that with a larger cavity a lower moisture content was needed to induce failure. The 2.4 m field cavity size for both the 3.8 cm and 2.5 cm thick laboratory samples had rapid collapse occur at slightly above optimum moisture content. The 3.7 m field cavity size for the 2.5 cm thick sample failed at 2 % below optimum moisture content.

Circumferential cracks at the upper surface and radial cracks at the lower surface were typical of most tests. Some of the low displacement (equilibrium) tests exhibited no cracking, with the tendency toward cracking stronger for the wider cavity opening for these samples. Many of the collapsed samples exhibited spalling of soil from the lower surface.

Finally, there did not appear to be an observable scaling factor for time from the tests. If this is true, then failure of the prototype cover would occur rapidly following subsidence, since this was the behavior exhibited by the laboratory models.

ACKNOWLEDGMENTS

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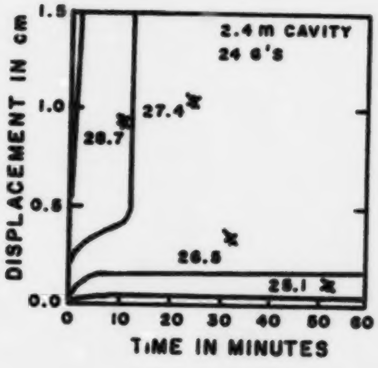

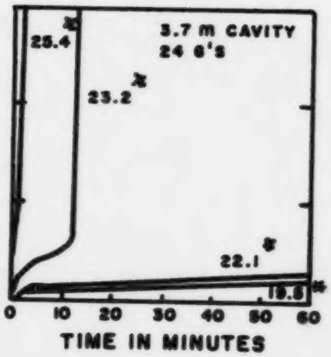

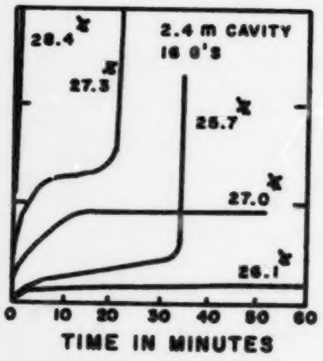

* PERCENT SOIL MOISTURE

Figure 5 (left). Responses of 2.5 cm thick clay layers to 2.4 m field size cavities being accelerated at 24 g's.
Figure 6 (center). Responses of 2.5 cm thick clay layers to 3.7 m field size cavities being accelerated at 24 g's.
Figure 7 (right). Responses of 3.8 cm thick clay layers to 2.4 m field size cavities being accelerated at 16 g's.

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LEACHATE COLLECTION SYSTEMS - SUMMARIZING THE STATE-OF-THE-ART

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ABSTRACT

A manual has been prepared to summarize the state-of-the-art in leachate collection system design, construction, inspection, maintenance and repair as it pertains to system failure. Leachate collection system failure mechanisms, as well as design, construction, inspection, maintenance and repair practices which may reduce the likelihood of failure, are reviewed.

INTRODUCTION

Leachate collection systems, the drainage systems which remove excess liquid accumulation above liners at land disposal facilities, like all other liquid collection systems, are subject to failure. Failure of leachate collection systems at hazardous waste disposal facilities are of particular concern since undetected failures may cause leachate to build up on top of the liner, which can lead to failure of the liner system and subsequent contamination of groundwater. Furthermore, failures which are detected may be difficult to repair, and replacement is not simple since excavation of hazardous wastes would be required.

In order to minimize the potential for system failure, it is first necessary to understand the mechanisms of failure and then to understand how these mechanisms can be avoided. As a first step toward this a manual was prepared which summarizes the state-of-the-art in leachate collection system design, construction, inspection, maintenance and repair as it pertains to the prevention and correction of failure. A schematic of a typical leachate collection system is given in Figure 1.

APPROACH

The Manual was prepared using information obtained from interviews with landfill owners and operators, design engineers, state and Federal regulatory officials, and equipment vendors, and from directly applicable and related literature. Information from the interviews included experience with leachate collection system performance and recommendations for system design, construction, inspection, maintenance or repair. Directly applicable literature mostly pertained to leachate collection system design and construction. Important related literature included materials on the inspection and maintenance of sewers and agricultural drainage systems.

Information collected was used in the Manual in two ways. First, experience with leachate collection system failure mechanisms found in the interviews and the literature was used to confirm that suspected failure mechanisms (1,3) are indeed possible in hazardous waste systems. Second, experience with leachate collection system performance, recommendations of experts, and information found in the literature were used to provide guidance on leachate

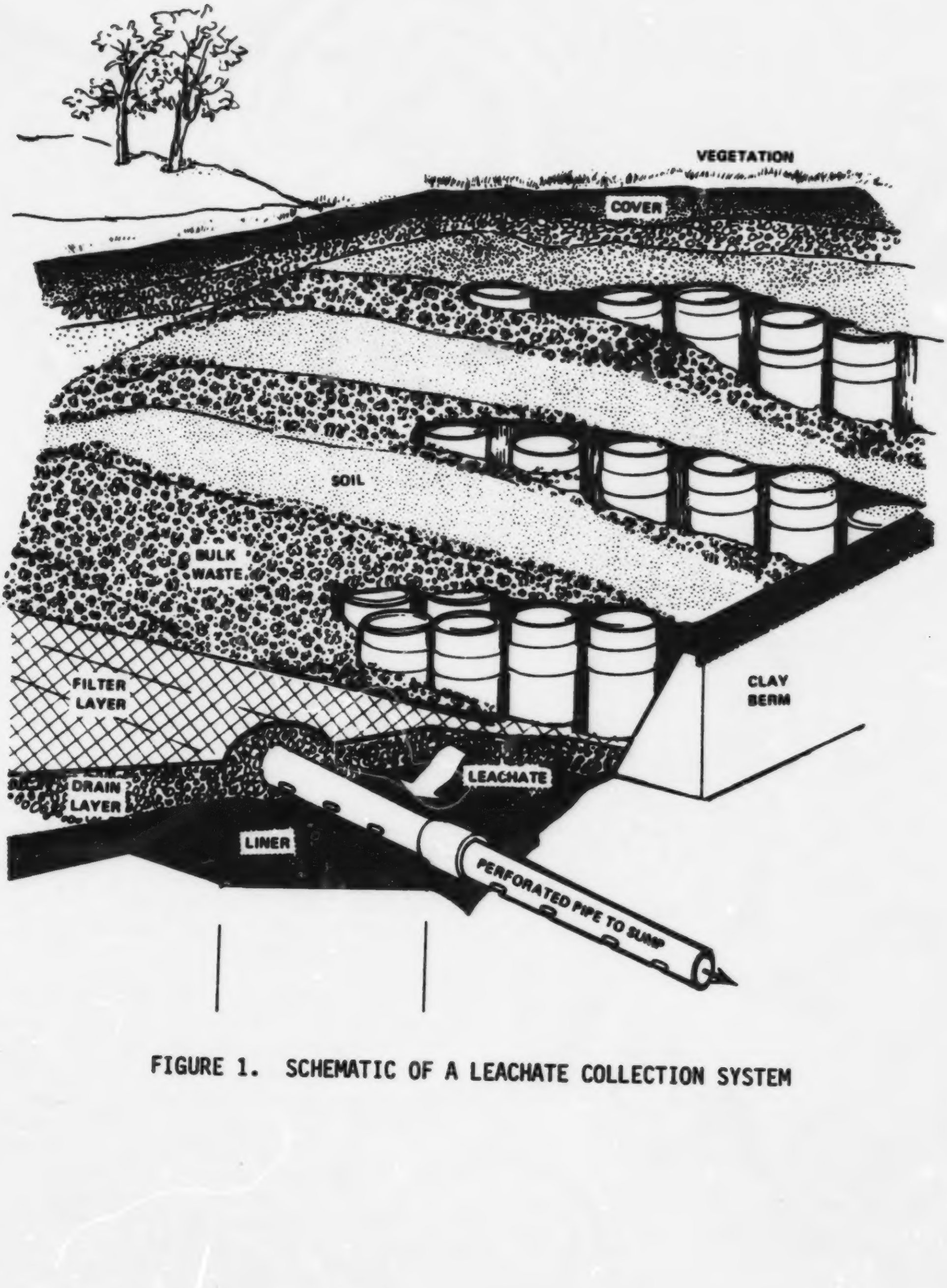
VEGETATION
COVER
SOIL
BULK WASTE
CLAY BERM
FILTER LAYER
LEACHATE
DRAIN LAYER
LINER
PERFORATED PIPE TO SUMP

FIGURE 1. SCHEMATIC OF A LEACHATE COLLECTION SYSTEM

collection system design, construction, inspection, maintenance and repair. Cap drainage systems were also discussed where appropriate.

RESULTS

Confirmation of Potential Failure Mechanisms

Table 1 provides a listing of 23 cases of failures which have been reported for actual leachate collection systems. Types of facilities included in the survey were municipal, co-disposal, industrial and hazardous waste landfills. When the facility type is not given or when the information is based on general experience from facilities of all types, the facility type in Table 1 is listed as "not specified". The cause listed in Table 1 refers to whether the failure was due to errors in design, construction, or operation. When a cause could not be determined based on the information provided in the interview, the cause is listed as being "undetermined". The final column in Table 1 provides comments on the failure mechanism noted. Three of the mechanisms are described with the phrase "not clogged". In these cases, the biological or chemical mechanism was active in the facility but a reduction in flow due to clogging was not noted.

Table 2 summarizes the experience with failure mechanisms by facility type. The table indicates that most of the experience is with municipal and co-disposal/industrial facilities and that only two examples were found at hazardous waste facilities. There are fewer hazardous waste than municipal and co-disposal faciilties to draw from, and companies may be less likely to provide information on problems in hazardous waste leachate collection systems. This observation alone does not demonstrate that failure is less likely to occur in hazardous waste leachate collection systems. Experience with failure mechanisms in municipal, co-disposal/-industrial and unspecified facilities may also be applicable to hazardous waste landfills if the cause of failure could occur regardless of facility type. Construction errors, for example, are independent of wastes to be handled, and can occur at any type of facility.

An analysis of the failure mechanisms indicates that all of the mechanisms are possible at hazardous waste disposal facilities; there is no conceptual basis for ruling out any of the mechanisms for hazardous waste leachate collection systems. Since the least experience was found for the biochemical mechanism, it was tested by conducting a limited laboratory study to determine the potential for biochemical clogging in a hazardous waste leachate environment. Results for this study indicate that a typical hazardous waste leachate does not appear to affect the formation of ochre in the leachate collection system. A detailed description of the laboratory program and results obtained are given in the Manual.

Design

Design considerations to prevent failure of leachate collection systems include pipe location, redundancy, maintenance features, and other features to address specific failure mechanisms.

Pipe location and placement is critical to the performance of leachate collection systems. Crushing or displacement due to equipment loading and differential settling is a well-recognized problem. A pipe is best protected when it is placed in a trench with careful consideration given to loading conditions and proper bedding. The trench provides added protection for the pipe, especially during placement of the first lift of waste when the pipe is most susceptible to crushing.

Redundancy in design is important to minimize the effect of failure mechanisms when they occur. The system should be able to remove liquid from any point in the facility by more than one pathway. One of the primary ways to provide redundancy is to design collection laterals such that drainage requirements can be met by the gravel layer alone if flow through the pipe is restricted. In addition, laterals should be spaced so that if one lateral is totally blocked, liquid can be removed through an adjacent lateral.

One of the most important considerations to prevent failure is designing

TABLE 1

LEACHATE COLLECTION SYSTEM FAILURES

Failure Mechanism	Facility Type	Cause	Comments
Pipe breakage	NS	O	by clean-out equipment if bends greater than 22°, general experience
Pipe breakage	municipal	D	differential settling, improper bedding
Pipe separation	municipal	C	joints not glued
Pipe deterioration	NS	D	problems with ABS pipe, general experience
Pipe deterioration	hazardous	O	from acid or solvent disposed of in wrong cell
Tank failure	co-disposal	D	leachate holding tank
Capacity exceeded	co-disposal	D	under-design, other problems noted
Capacity exceeded	hazardous	O	periodic rather than automatic pumping of sump
Outlet inadequate	co-disposal	D	caused leachate buildup
Sedimentation	NS	C	no filter installed
Sedimentation	NS	U	general experience
Sedimentation	co-disposal	U	in 1 year old system
Sedimentation	co-disposal	U	of gravel layer and pipe
Sedimentation	municipal	U	general experience
Sedimentation	NS	C	general experience
Biological growth	industrial	D	100 ft. long biological growth flushed out under high pressure
Biological growth	municipal	U	reduction in flow every 2 years; flushed out
Biological growth	municipal	U	of filter fabric
Biological growth	co-disposal	U	on 3/4 inch stone, not clogged
Chemical precipitation	municipal	O	EPA test cell, not clogged
Chemical precipitation	co-disposal	U	iron oxide, not clogged
Chemical precipitation	co-disposal	O	attributed to waste characteristics
Biochemical precipitation	co-disposal	U	in leachate collection wells

NS = not specified; C = construction related; U = undetermined; O = operation related; D = design related.

TABLE 2
SUMMARY OF FAILURES BY FACILITY TYPE

Mechanism	Municipal	Co-disposal/ industrial	Hazardous	Not Specified	Total
	Facility Type				
Pipe breakage	1	-	-	1	2
Pipe Separation	1	-	-	-	1
Pipe deterioration	-	-	1	1	2
Other non-clogging problems	-	3	1	-	4
Sedimentation	1	2	-	3	6
Biological growth	2	2	-	-	4
Chemical precipitation	1	2	-	-	3
Biochemical precipitation	-	1	-	-	1
TOTAL	6	10	2	5	23

the system to facilitate inspection and maintenance. Access should be provided to all parts of the system. This includes the placement of manholes and cleanouts so that maintenance equipment can reach any section of pipe. The design should consider minimum pipe size, distance between access points, and maximum angles negotiable by such equipment. For example, some jet cleaning equipment requires at least a four inch pipe and can operate in straight runs up to 1000 feet in length. In one case, cleanout equipment is reported to break pipes at bends greater than 22 degrees.

Finally, leachate collection systems can be designed to avoid specific clogging mechanisms. For example, sedimentation can be avoided by selecting the proper grain size distribution for the filter material, and by providing minimum slope and avoiding hydraulic perturbations to maintain flow velocity during leachate collection so that solids can not settle out. Maintaining flow velocity will also help prevent buildup of biological and chemical materials. Selection of construction materials based on the wastes to be handled by each cell of a hazardous waste disposal facility can help avoid deterioration caused by reactions with waste leachates.

Construction

Leachate collection system construction according to design specifications is critical. Specific construction problems which can cause leachate collection system failure include:

- the use of materials with specifications other than the ones in the approved design;
- foreign objects (e.g., rocks, clumps of soil, construction tools) left in drainage system piping which plug or restrict leachate flow and may not be

removable using designed maintenance procedures;

- neglecting to install materials in the location specified in the design;
- siltation of filter or drainage layers resulting from improper upgradient drainage during construction and/or careless construction techniques; and
- the use of improper construction equipment or improper placement procedures which cause damage to system components, such as crushing or misalignment of pipes due to loading of heavy equipment.

Construction quality assurance (CQA) is needed to verify that the completed leachate collection system meets or exceeds the design requirements. This involves monitoring and documenting the quality of materials used and the conditions and manner of their placement. CQA serves to detect variations from design, whether as a result of error or negligence of the contractor, and to provide for suitable corrective actions before wastes are accepted at the facility. Without proper CQA, problems with leachate collection systems due to construction, like the two specific examples noted in Table 2, would not be discovered until the system failed during operation.

In addition to CQA, specific construction methods may be important in avoiding failure mechanisms. For example, John Pacy of Emcon Associates (2) recommends placing pipe in trenches in small sections and completing each section before starting on the next in order to minimize the amount of surface sediments which is washed into the drain during construction. Construction techniques which are important to system performance should be specified in the design and monitored by the CQA program.

Inspection

Inspection methods are intended to determine whether failure mechanisms are active in the leachate collection system. Regular inspections include monitoring flow at outlets or access points, monitoring leachate level within the facility, correlating leachate quantity with rainfall data, and correlating leachate quality with clogging indicators, such as the presence of iron reducing bacteria. Direct methods include TV or photographic inspection of drainage pipes and checking the physical continuity of the pipes with sewer cleaning equipment. Direct methods are limited to use in drain pipes, since the drainage and filter layers are not accessible to physical inspection.

In addition to regular or periodic inspections, the following special inspections should be performed as appropriate:

- as-built inspection, as part of the CQA plan, to verify continuity of the pipe network after construction.
- re-verification of continuity after the first lift of waste is placed.
- testing to locate and diagnose suspected problems.

Maintenance

Experience with maintenance techniques at leachate collection systems is lacking. It seems that system repair is generally used as a substitute for maintenance. Although maintenance and repair often involve the same methods, regular maintenance may be the more cost-effective option, since a less intensive effort may be required and since some problems which can be avoided by maintenance cannot be easily corrected once they begin.

The simplest maintenance technique is flushing of the pipe network. This can be done by gravity flow using water or leachate, or by low pressure jet cleaning equipment. Flushing and jetting remove sediments and immature precipitate before substantial buildup can occur. Standard sewer cleaning devices such as roto-rooters, snakes, pigs, buckets, sewer balls may also be used to clean the collection system. These perform basically the same function as flushing and are also a good check on the continuity of the drains since they can discover crushed or displaced pipes, as well as areas of more serious clogging.

Control of some failure mechanisms can also be accomplished by controlling waste inputs to the extent possible. Wastes placed in the first lift, for example, should have a high permeability and may have special chemical characteristics. High permeability wastes are placed in the first lift to facilitate the flow of leachate to the collection system. An example would be a carefully placed layer of upright drums with sand or soil placed in between the drums. Special waste characteristics in the first lift may include wastes of relatively lower pH, wastes which inhibit biological activity, and wastes which do not contain high iron, calcium, magnesium, nutrient, or sediment content. The chemical characteristics desirable in the first lift of waste are dependent on the leachate characteristics expected in general at the facility, and the failure mechanisms which are expected to be active.

Currently, no techniques are available for the maintenance of drainage or filter layers. Certain chemical techniques developed for other drainage systems (SO_2 gas, hydrogen perioxide, acid treatment) may be applicable to drainage layers in leachate collection systems, although considerable testing would be required. Compatibility of the chemicals with the waste disposed of at the facility must also be considered.

Repair

Repair of a leachate collection system is necessary when the system is unable to remove leachate from the facility, causing leachate to accumulate over the liner. The need for repair can be determined using the inspection techniques discussed above. If the failure of the leachates system is due to clogging of the pipe, standard sewer cleaning techniques may be used to correct the problem, provided the pipe is accessible via manholes or cleanouts.

When the pipe is not accessible or when the problem is with the drain or filter layer, or when maintenance techniques are not effective, replacement of the failed portion of the leachate collection system may be required. Replacement techniques include:

- excavating the failed portion with repair or replacement using new materials. The acceptability of excavation will depend on the type of waste in the facility (to evaluate safety hazards posed by excavation) and the number of lifts of waste in place.
- retrofitting a new collection line adjacent to the failed portion to remove accumulated leachate.
- installing well points through the waste to remove accumulated leachate.

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VERIFICATION OF LEACHATE COLLECTION EFFICIENCY
WITH PHYSICAL MODELS

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ABSTRACT

Two large-scale physical models of landfill liner/drain systems were constructed to examine the effects that the length, slope and hydraulic conductivity of the drain layer and the depth of saturation above the liner have on the subsurface lateral drainage rate. The models have two different lengths and adjustable slope. The models were filled with a 3-foot sand drain layer overlying a 1-foot clay liner. A 2-inch layer of gravel was placed under the liner to collect seepage from the clay liner. The design, construction, instrumentation and preparation are described in detail.

Several drainage tests were run on each configuration of the models by applying water as rainfall to the surface of the sand layer and then measuring the water table along the length of the models and the drainage rate as a function of time. The experimental design and test procedures are presented along with some descriptive results.

INTRODUCTION

Background

Landfills have been widely used for disposal of municipal, industrial and hazardous solid wastes. Storage of any waste material in a landfill poses several potential problems. Among these is the possible contamination of ground and surface waters by the migration of water or leachate from the landfill to adjacent areas. Given this potential problem, it is essential that the liquids management technology perform as expected during its development. It is also essential that the performance of the technology can be simulated or modeled with sufficient accuracy to design landfills to prevent migration of water from the facility. The modeling of the moisture movement through landfills is also important to review landfill designs and to determine the adequacy of the design and the limitations of the liquids management technology.

This study was conducted to test and verify the liquid management technology of lateral sub-surface drainage in covers and leachate collection systems. Specifically, this involves verification of the Hydrologic Evaluation of Landfill Performance (HELP) Model and other regulatory and technical guidance provisions and procedures developed by the U.S. Environmental Protection Agency (USEPA).

The HELP model is a computer model that generates water budgets for landfills by performing a daily sequential simulation of water movement into, through and out of a landfill. The model provides estimates of runoff, evapotranspiration, lateral drainage, percolation and depths of saturation. Lateral drainage is computed in the model as a function of the average depth of saturation above the liner, the slope of the surface of the liner, the length to the drainage collector, and the hydraulic conductivity of the lateral drainage layer (1, 2):

$$Q = \frac{2(0.51 + 0.00205\alpha L)K\bar{y}\left(\bar{y}\left(\frac{\bar{y}}{\alpha L}\right)^{0.16} + \alpha L\right)}{L^2}$$

where:

Q = lateral drainage rate, inches/day (in/day)
$\underline{K}$ = hydraulic conductivity, in/day
$\bar{y}$ = average depth of saturation, inches
α = slope, dimensionless
L = drainage length, inches

The USEPA regulatory provisions for lateral drainage layers require only that the depth of leachate buildup at the bottom of the landfill should not exceed 1 foot (ft) and the construction materials for leachate collection should be resistant to chemical attack and the physical forces exerted on them (3). USEPA technical guidance states that the drainage layer be constructed to be at least 12 inches thick at a minimum slope of 2 percent and to have a hydraulic conductivity of not less than 1×10^{-3} centimeters/second (cm/sec). Also, the drainage tile system should be of appropriate size and spacing to efficiently remove the leachate. It is believed that 4-in diameter (dia) tiles spaced 50 to 200 ft apart would be adequate for removing leachate (3). The drainage layer in the cover should have the same specifications as above except that tile drainage systems are not necessary although free drainage must be provided at the perimeter of the cover (3).

Purpose

The objective of this study was to verify the USEPA technical guidance by determining the lateral drainage rate as a function of the hydraulic conductivity, slope, length and depth of saturation of the lateral drainage layer in large-scale physical models. The measured average depths of saturation, drainage rates and drainage times in the physical models will be compared with the results predicted by the HELP model and its equations for vertical and lateral drainage. For the remainder of this paper the term, "model" refers to the physical models.

EXPERIMENTAL APPROACH

Model Design and Construction

Two physical models were designed and constructed to perform the laboratory verification tests. The models were constructed at two different lengths to permit the examination of the effects of length on drainage. One model was built to have a usable depth of 5.0 ft, an inside width of 5.3 ft, and a drainage length of 25.4 ft. The other model had the same depth and width but had a drainage length of 52.4 ft.

Both models were constructed of identical materials. The base of the models were constructed of steel soil test cars for mobility studies. These cars are reinforced 1/4-in steel tanks that are 27 ft long, 5.3 ft wide and 2.5 ft deep. The sides of the cars were extended upward 4 ft with marine grade, 1/2-in plywood supported by 2-in angle iron. Silicone sealant was used to fill the joints and cracks and to seal and prevent leaks.

The models were placed at a two percent slope and fitted with supports for attaching 5-ft screw jacks at the upper end. A jacking structure was built to attach to either model which could raise the end of a fully loaded model. The jack could increase the slope of the long model to eleven percent and the short model to twenty percent. The slopes of the models during testing were determined by surveying.

Three troughs, approximately 12 inches deep and extending across the width of the models, were placed inside the lower end of each model. The bottom trough was used to collect seepage through the clay liner. The middle trough collected subsurface lateral drainage at the bottom of the sand drainage layer. The top trough collected runoff from the surface of the sand. The bottom two troughs were constructed of 1/4-in steel plates welded to the steel soil test car. The top trough was fabricated of galvanized sheet metal and bolted in place to the plywood walls. The sheet metal was sealed to the walls with silicone sealant.

The runoff trough was drained to a 5-ft deep, 32-in dia sump tank via 1 1/2-in polyvinyl chloride (PVC) pipe. The water level in the tank was measured with a steel measuring tape before and after a rain event to determine the runoff volume. The drainage trough was also drained to a 5-ft deep, 32-in dia sump tank via 1 1/2-in PVC pipe. The drainage sump tanks were automatically pumped out when filled to a height of about 44 inches, requiring only about 5 minutes to lower the water depth to about 10 inches. The minimum time required to fill the tank was about 4 hours. The seepage trough was periodically drained into a 4-litre graduated cylinder to measure the seepage volume. The seepage was drained from the trough via an attached 2-in rubber hose which was sealed between measurements.

The lateral drainage rate from the sand layer was determined by two devices, one for low flow rates and the other for high flow rates. Low flow rates (below 0.08 gallons/minute (gpm) or below 0.05 inches/hour (in/hr) for the short model and 0.02 in/hr for the long model) were determined by measuring the drainage volume as a function time using a Weathertronics Model 6010 Tipping Bucket Rain Gage. The tipping bucket collected the water from the end of the PVC pipe draining the drainage trough and discharged the water into the drainage sump tank. A bypass was placed in the PVC pipe to divert the drainage directly into the sump tank during periods of very high flow rates. High flow rates (greater than 0.08 gpm) were determined by measuring the water level in the drainage sump tank as a function of time using a Weather Measure Corporation Model F553-A Water Level Recorder powered at 12 volts by a ELCO Model 1060S Low Ripple Battery Eliminator and Charger.

Both models were fitted with a device to spray water uniformly across the length of the surface of the sand drain layer. This device consisted of a 1 1/2-in PVC pipe mounted on a 12-ft long aluminum cart that tracked back and forth on rails that ran the length of the model. A nozzle that sprayed a uniform line of water across the width of the model was attached on each end of the pipe. Each nozzle sprayed on one half of the model's sand surface as the cart moved its maximum distance in one direction. The height of the nozzles and the travel limits of the cart were adjustable to insure that the entire surface of the sand was sprayed. The water application rate or rainfall intensity was adjusted by changing the size of the nozzles or changing the water pressure applied on the nozzles. The intensities of flow rates could be varied between 0.02 in/hr to 6 in/hr or 0.06 gpm to 10 gpm. The intensity during a test was determined by measuring the volume of water applied as a function of time. A stopwatch was used to measure the time and a Badger Meter, Inc. Model Recordall 12 water meter measured the water volume. The nozzles used in the tests were Floodjet nozzles 1/8 K.25 316SS through 1/4 K18 316SS that were manufactured by Spraying Systems Co.

The major features of the models are shown in the schematic in Figure 1. The schematic is a side view of the short model showing the loaded test cell, troughs, drain lines, sumps, jacking structure, rain cart, and instrumentation.

Test Preparation

After construction was completed, the models were filled with three functional layers for testing and a layer of fill dirt as shown in Figure 1. The top layer was a 3-ft sand layer for lateral drainage. The second layer was a 1-ft compacted clay liner designed to minimize percolation. The third layer was a 2-in layer of pea gravel to transmit the seepage from the clay liner to a drain. The bottom layer was a 10-in layer of fill dirt to build up the pea gravel layer to promote drainage.

Ten inches of fill dirt was compacted in the bottom of the steel test car to fill the trapezoidal section of the model and to reduce the volume of pea gravel required. The fill dirt was covered by a 30-mil, butyl rubber impermeable membrane, T-16 manufactured by Firestone. The membrane was glued to the walls of the steel car to prevent percolation into the fill dirt. G-580, a synthetic rubber resin dispersed in solvent and manufactured by Pittsburgh Paint & Glass, was used as the glue.

Two inches of washed creek pea gravel was placed on the impermeable membrane and in the seepage collection trough. This layer was designed to have a high permeability and low storage potential in order to transmit the small volume of seepage rapidly to the collection trough. The layer was covered with Bidim Type C-34 filter fabric manufactured by Monsanto to prevent migration and penetration of the clay liner into the pea gravel and to protect the ability of the pea gravel to transmit seepage. The filter fabric was glued to the walls of the steel car to insure that the fabric and pea gravel did

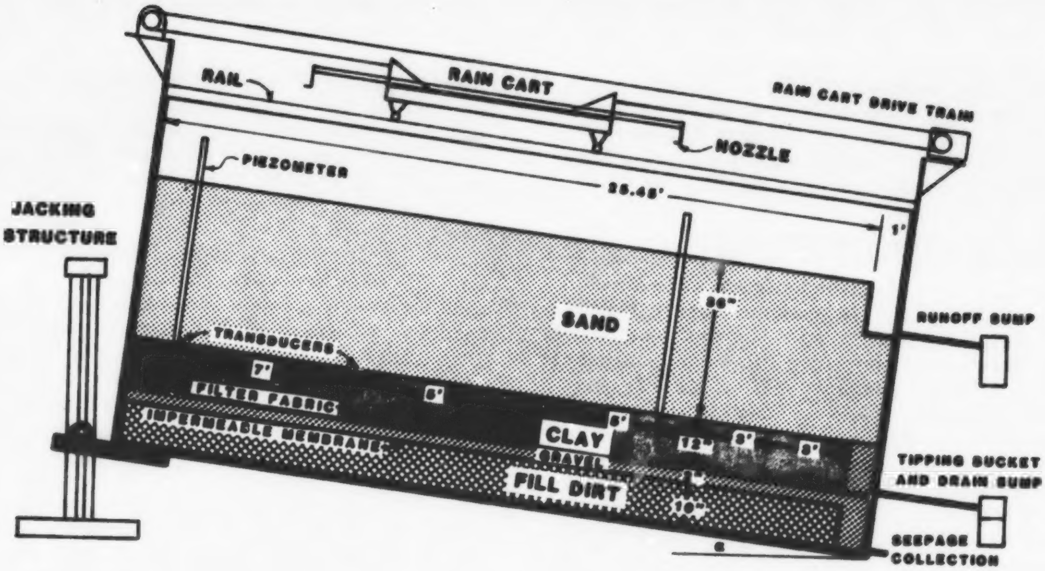


Figure 1. Schematic of the Short Model.

not move during placement of the clay liner. Both the gravel and fabric were wetted during placement of the liner. Wetting reduced the seepage required to sufficiently deplete the storage volume of the pea gravel and rubber membrane for transmission of the seepage to begin.

A 12-in clay liner was placed above the seepage transmission layer. Buckshot clay, a locally found, well-defined clay, was used. The clay was spread on a concrete strip and cut up into small clods using a scarifier. The clay was then wetted to a moisture content of about 27% and placed in the model in two-inch lifts. Each lift was compacted with a gasoline-powered, hand-operated compacter manufactured by Wacker. The compacter had a compacting area of about 0.5 square feet (sq. ft). Additional water was added during compaction to insure good blending and to prevent drying. At least four passes were made on each lift with the compacter at which time no additional compaction could be discerned by additional passes.

After placement of the clay liner was completed, a sample of the top 3 inches was taken in the short model. Its water content was 27.1%, its dry density was 94.9 pounds per cubic foot (pcf) and its coefficient of permeability was 1.67 x 10^{-7} cm/sec. A second sample, taken at a depth of 6 inches while filling, had a moisture content of 28.3% and a dry density of 91.0 pcf. Nuclear density probe measurements indicated a moisture content of 34.5% and a dry density of 89 pcf. The moisture content read by the nuclear density meter is much higher because it includes water of crystallization and hydration which would not be and was not measured by the standard moisture content test. Buckshot clay is known to have a high content of bound water. In all of the samples the dry densities were very high for the moisture content which indicates very good compaction. The soil testing were performed in accordance with ASTM accepted procedures as outlined in Laboratory Soil Testing, EM 1110-2-1906 (4). The nuclear density measurements were made with a Troxler Electronic Laboratories, Inc. Model 3411B nuclear density gage using the procedures outlined in its instruction manual. The same sampling procedures were used for the long model with similar results.

While placing the clay liner, a flap of the T-16 rubber membrane was glued to the walls of the steel car at eight inches above the base of the clay and four inches below the eventual top of the clay liner. This flap extended four inches into the clay. The purpose of the flap was to prevent seepage from occurring between the car and the clay by diverting the water into the clay in the event that the clay shrank or cracked at the walls.

Six transducers were placed in the long model and five in the short model. Consolidation Electrodynamics Corporation Type 4-312 pressure pickups were fitted in a brass body with a porous stone such that the pressure transducer could only measure pore pressure which in this case was the head of water on the transducer. The transducers were placed at the top of the clay liner and were exposed to the sand layer. The locations of the transducers were approximately 3, 6, 11, 16 and 23 ft from the lip of the drainage trough in the short model and 3, 6, 11, 23, 35 and 50 ft from the trough in the long model. The transducers were placed on the centerline dividing the width of the models. The elevations and locations of the transducers were determined by surveying with an optical transit.

A 3-ft layer of sand was placed above the clay liner. The sand was placed in 4-in lifts and each lift was vibrated until no additional compaction could be discerned. In general, four passes were made with a hand-operated, gasoline-powered sand compacter manufactured by Wacker. The size of the vibrating plate on the compacter was about 2 sq ft.

Two different sands were used in separate tests. One sand was Reid-Bedford sand with about ten percent Windham clay coarsely dispersed throughout the sand in the short model and with about thirteen percent clay in the long model. The dry density, as placed, was 106 pcf, the specific gravity was 2.7, and the laboratory coefficient of permeability was about 4×10^{-3} cm/sec. The porosity was 0.37, the drainable porosity was about 0.21 to 0.25, and the wilting point (i.e. the moisture content at 15 bar tension) was about 0.07. The effective drainable porosity was much smaller due to air being trapped in the sand. The volume of air varied during testing in the physical model.

The other sand was a locally found, ungraded washed creek sand. The dry density, as placed, was 107 pcf in the long model and 110 pcf in the short model. The specific gravity of the sand was 2.65, and the coefficient of permeability was 2.2×10^{-1} cm/sec. The porosity was 0.35 in the short model and 0.36 in the long model. The drainable porosity was about 0.24 to 0.28 and the wilting point was about 0.04 to 0.05.

Piezometers were installed in the sand layer to provide a backup method to measure the depth of saturation above the clay liner. These measurements were also used to calibrate the pressure transducers and to check the accuracy of the transducers throughout the testing. Piezometers were constructed by attaching 1 1/2-in diameter, 4-in long cylindrical porous stones to 1/2-in PVC pipes of approximately 4-ft lengths. The piezometers were installed with the stone end placed flush with the top of the clay liner and with the pipe extending straight up through the sand layer. The depth of saturation was determined by measuring the depth to the water surface from the end of the pipe and then subtracting that value from the total length of the piezometer. Piezometers were placed at the locations of the second and last transducers from the lip of the drainage trough in the short model and at the second, fourth and last transducers from the lip of the drainage trough in the long model.

Experimental Design

A complete block experimental design was used to examine the effects of drainage length, slope, hydraulic conductivity, depth of saturation, rainfall intensity and rainfall duration on the lateral subsurface drainage rates. The block design was selected because it provided the most data with the least time and expense for construction and model preparation. Only two models could be built due to their costs and only two sands could be examined due to the unavailability of another of significantly different permeability. Several slopes and rainfall events could be examined quickly since very little time was required for changing these test conditions. Also, the time requirements and costs for running an additional test with a different slope or rainfall were less than ten percent of the requirements for preparing the model for a different sand. Additional rainfall events were examined in lieu of replicates since the lateral drainage rate as computed by the HELP model does not directly consider the effects of rainfall intensity or duration. Also, since a complete block design was used, the effect of a change in a variable is directly examined under multiple test conditions, reducing the need for replicates.

In the model tests two drainage lengths were compared -- 25.4 ft and 52.4 ft. Three slopes were examined -- approximately 2, 5 and 10 percent. Sands of two hydraulic conductivities were used -- 4×10^{-3} cm/sec and 2.2×10^{-1} cm/sec. Four rainfall events were examined -- a 1-hr rainfall at 0.50 in/hr, a 2-hr rainfall at 1.50 in/hr, a 6-hr rainfall at 0.50 in/hr and a 24-hr rainfall at 0.125 in/hr. In addition to these drainage tests, the sand was saturated, predominantly from the bottom up for several test conditions, and then allowed to drain. Also, water was applied to the sand for a long period of time (generally more than 36 hours) at a rainfall intensity which would maintain the average depth of saturation in the sand at 12 inches.

Test Procedure and Data Collection

Each test was run in one of three manners. Lateral drainage was measured either from a pre-saturated sand, from the sand before, during and after a rainfall event, or from the sand while attempting to maintain a constant average depth of saturation or head at twelve inches. The test procedure for the three manners were very similar; only the procedure for applying the water to the model varied.

The procedures for running a test were as follows:

1. Prepare model for testing. This included any preparation of the sand such as placing or leveling, preparation of rainfall cart for desired rainfall intensity by changing nozzles, nozzle heights and end stops, changing the slope of the model, and draining sump tanks.

2. Initialize test which included starting the recorders for drainage and heads, setting the time, draining the lateral drainage trough, recording test conditions, measuring initial depth of water in runoff sump tank and recording the initial water meter reading.

3. Apply water to the model as prescribed for the test. This included adjusting the water application rate and the spray pattern.

4. Note peculiarities of the test including unusual environmental conditions, problems, and variations from the test procedure.

5. After the rainfall was completed and runoff ceased, measure the final depth of water in runoff sump tank.

6. Measure water levels in piezometers periodically.

7. Drain tanks as required.

8. Continue testing until the heads become so low as to nearly expose a transducer to air and then stop drainage or start a new test.

9. Collect data from recorders and reduce the data.

Prior to calibration of the pore pressure transducers, water was added slowly to both ends of the model. In this manner the sand was saturated primarily from the bottom up which forced the air out of the sand. Following the calibration, the sand was drained and the drainage rate and heads on the liner were measured continuously while draining. Drainage from pre-saturated sand was measured for only four test conditions since saturation was required only for calibration.

For the tests in which a constant average head of twelve inches was maintained, water was applied to the model to increase the head on the liner rapidly to twelve inches. This water was applied by raining at an intensity of about 0.5 in/hr. After reaching a head of twelve inches, the rainfall intensity was decreased to the intensity estimated to maintain the head. The desired intensity was determined by adding together the evaporation rate, the rate of leakage and seepage, and the drainage rate which corresponded to an average head of the twelve inches during a previous rainfall for the same physical test conditions. The intensity was adjusted during the test if the head did not remain constant near twelve inches. The rain was continued for at least 12 hours after the head remained nearly constant.

The evaporation rate ranged from about 0.2 to 0.3 in/day while raining. The rate was dependent on building temperature and on the spray nozzle. The building temperature ranged from about 58°F to 90°F but for most tests the temperature ranged between 70°F and 82°F.

Leakage varied directly as a function of head on the liner. Both cars leaked but the short model was more watertight than the long model. The leakage rates for the short model were about 0.02 in/day at a head of 20 inches and about 0.01 in/day at a head of 12 inches while the leakage rates for the long model were about 0.12 in/day at a head of 20 inches and about 0.07 in/day at a head of 12 inches. Neither model leaked at heads below 6 inches since all of the seams in the model were above this depth. The long model leaked more since it suffered greater deflections upon loading, causing more seams to rupture and leak. The leakage rates, although important from an overall water balance perspective, have a negligible influence on the drainage rates.

The majority of the drainage tests encompassed measurements before, during and after a specified rainfall event. In these tests, a specified quantity of water, either 48.5 or 291 gallons for the short model and either 98.5 or 591 gallons for the long model, was sprayed across the entire surface of the sand in a specified period of time, either 1, 2, 6 or 24 hours. These quantities of water correspond to about 0.57 inches and 3.4 inches of rain, respectively for both models. The tests measured the heads and drainage rates as the head increased from a starting point of about 4 inches to a peak value occurring about 30 minutes after the rainfall ceased and then returned to about 4 inches when the test was stopped. Several tests were allowed to continue until the head was nearly zero inches.

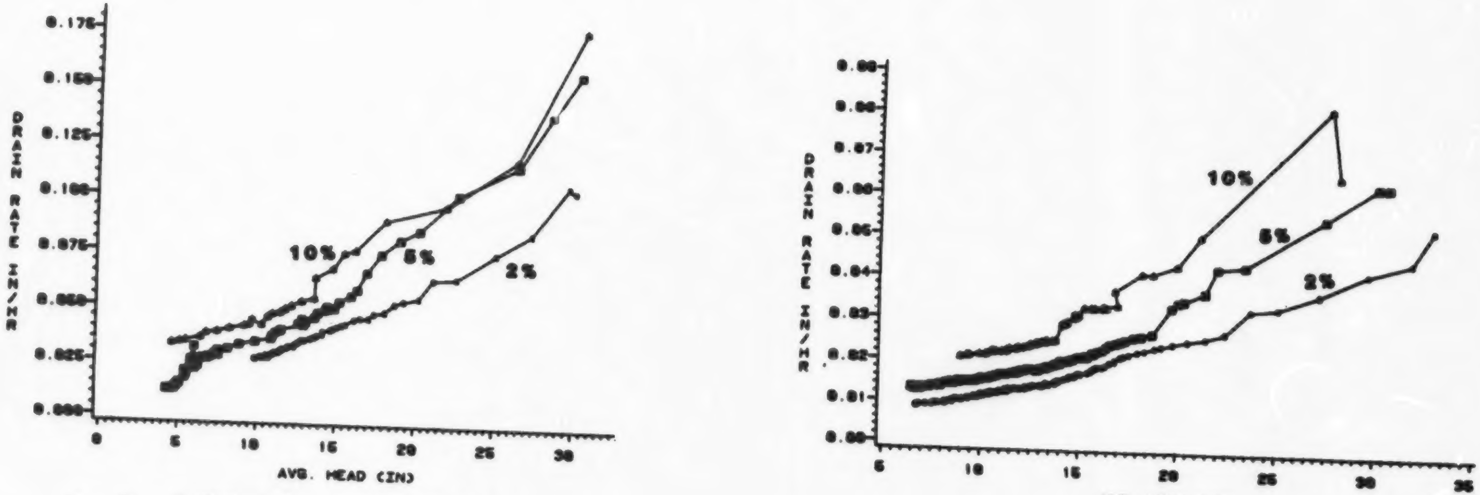
RESULTS

Twelve model configurations were examined in this study -- two different sands placed in two models of different length (25.4 ft and 52.4 ft) and tested at three slopes. Typical results of the drainage tests for these configurations are presented in Figure 2. Each curve is a plot of the drainage rate per unit area as a function of the average depth of saturation above the liner and shows results for drainage from a model with a given drainage length and hydraulic conductivity at three slopes -- 2, 5 and 10 percent. The results shown here are from tests using a rainfall duration of 6 hours and a rainfall intensity of 0.5 in/hr except for the curve for 2 percent slope in Figure 2a. This curve shows the results for a rainfall duration of 24 hours and a rainfall intensity of 0.125 in/hr. The results for other durations and intensities are similar to those shown here.

Figure 2 shows the effects that independently changing each of the four major parameters (head on the liner or depth of saturation, slope, hydraulic conductivity and drainage length) has on the drainage rate. Each graph shows the effects of changes in slope and head on the liner while holding the other parameters constant. Comparisons between Figures 2a and 2b and between Figures 2c and 2d show the effects of changes in drainage length. Comparisons between Figures 2a and 2c and between Figures 2b and 2d show the effects of changes in hydraulic conductivity.

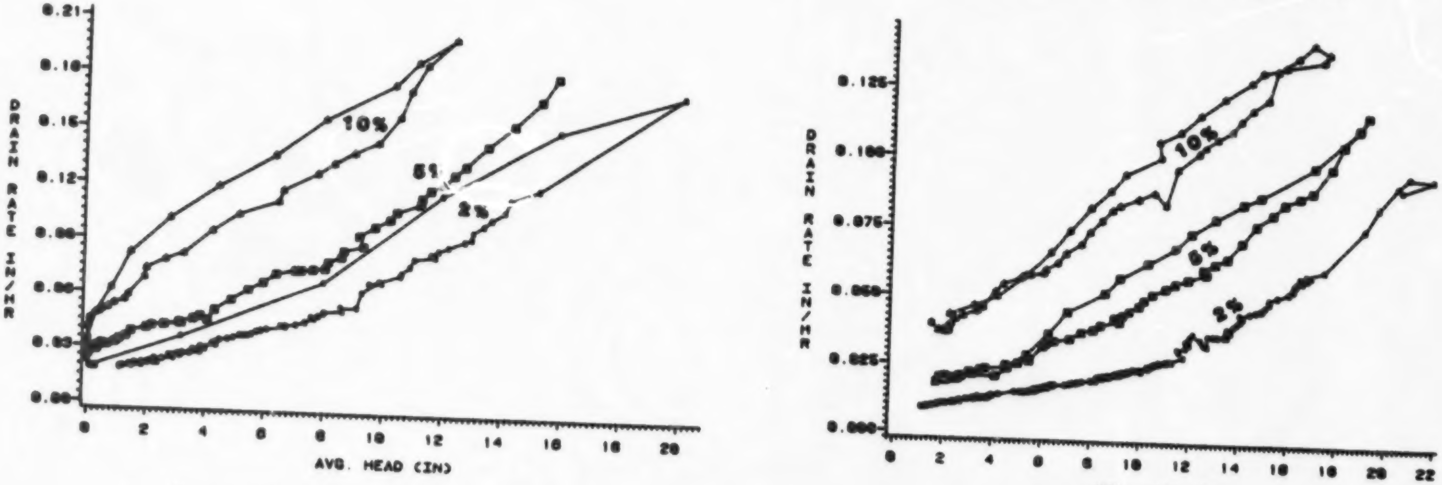
Figures 2a and 2b show the results for fine sand in the short model and in the long model, respectively. The permeability was measured in the laboratory to be about 0.005 cm/sec in the short model and 0.004 cm/sec in the long model but the results indicate that the permeability of the sand as placed in the models was higher. The permeability of the sand was about 0.006 to 0.007 cm/sec in the short model and about 0.007 to 0.009 cm/sec in the long model.

Figures 2c and 2d show the results for coarse sand in the short model and in the long model, respectively. The permeability of the coarse sand was measured to



a. K = 0.004 cm/sec L = 25.45 ft

b. K = 0.004 cm/sec L = 52.45 ft



c. K = 0.22 cm/sec L = 25.45 ft

d. K = 0.22 cm/sec L = 52.45 ft

Figure 2. Curves of Drainage Rates as a Function of Head on the Liner and Slope

be 0.22 cm/sec in the laboratory but the results in the model indicate that permeability was much lower. The results for the short model indicate that the permeability was about 0.017 to 0.025 cm/sec while the results for the long model indicate about 0.023 to 0.030 cm/sec. The results show an hysteresis indicating that the drainage rate was greater while the head was building than while falling.

Detailed examination of the results has not been completed at this time; however, preliminary results indicate that, as expected, an increase in the head on the liner, slope or hydraulic conductivity yielded an increase in the drainage rate per unit area of surface as did a decrease in the drainage length. The changes were similar in magnitude to those predicted by the equation used in the HELP model when using the apparent hydraulic conductivity of the sand as placed instead of the value measured in the laboratory. The agreement between the hydraulic conductivity of the sand measured in the laboratory and the apparent value determined from the drainage results was poor.

SUMMARY

Two large-scale physical models of landfill subsurface drainage systems were designed and constructed to have drainage lengths of 25.45 ft and 52.45 ft. The models were filled with a 1-ft clay liner and a 3-ft layer of sand. Drainage tests were conducted on two sands having permeabilities of 0.004 cm/sec and 0.22 cm/sec. The models were tested at slopes of 2, 5, and 10 percent. The analysis of the results was not completed at the submittal time of this paper but the initial examination of the results indicated that the permeability of the sand as measured in the laboratory did not agree well with the apparent hydraulic conductivity of the sand as placed in the model. The results showed the expected trends where an increase in the head on the liner, slope or hydraulic conductivity yielded an increase in the drainage rate per unit area of surface as did a decrease in the drainage length.

DISCLAIMER

Mention of trade names or commercial products does not constitute endorsement or recommendation for use by the U.S. Environmental Protection Agency or the U.S. Army Engineer Waterways Experiment Station.

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CHEMICAL RESISTANCE OF FLEXIBLE MEMBRANE LINERS IMMERSED IN SELECTED AQUEOUS SOLUTIONS

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ABSTRACT

The goal of this project is to develop screening criteria for evaluating the chemical compatibility of flexible membrane liner (FML) materials. These criteria will be used to make a material/chemical waste compatiability guide.

Data on the resistance of five common FML's to nine different chemicals plus water are being generated during the project. The data will be compiled and compared with existing chemical resistance tables (from trade groups and manufacturers) to form a general matrix for screening FML chemical resistance. The use of accelerated test procedures (high temperature immersions) will be evaluated by testing at 23°C and 50°C.

The FML materials represent the three major types of liner materials: elastomers (ethylene propylene rubber and epichlorohydrin); thermoplastics (polyvinyl chloride and chlorinated polyethylene). Unsupported materials only are being are used to focus the study on polymer/chemical interaction.

The nine chemicals are sodium chloride, hydrochloric acid, sodium hydroxide, potassium dichromate, methyl ethyl ketone, phenol, furfural, ASTM #2 oil, and 1,2-dichloroethane. Each represents a chemical type commonly found in waste streams. Although an actual waste stream will usually be a mixture of several components, this study uses only one chemical per exposure in water solution. Inorganic chemicals represented are solutions of acid, base, salt, and oxidizer. Organic groups represented are ketone, aldehyde, organic acid, oil, and chlorinated hydrocarbon. Some of the specific chemicals were chosen to correspond with those being used in related FML chemical resistance studies.

Initial screening showed that most materials increased in weight and volume during a seven day exposure period, but in many cases returned to very near the original size and weight after the air-drying period. It was decided to test materials as soon as possible after immersion and to keep them from drying out before being tested. Some of the materials were destroyed by pure solvents, as expected. As a result, the majority of concentrations chosen for further testing were at or below the water solubility limit. Three concentrations of each organic chemical were selected to show the effect of concentration on material change.

Samples are tested for change in weight, volume, tensile properties, and tear resistance after exposure times of 1, 7, 14, 28, and 56 days at each temperature. Also, weight and volume change in material samples at each chemical/temperature combination are measured every four months in a two year exposure test. The samples are also visually inspected for excessive degradation. Where possible, at the end of the two year test period, the tensile properties of the exposed samples will be measured.

Data from these immersion tests are studied to determine the correlation between different types of measurements and tests, and to identify major indicators of membrane degradation. The effect of chemical type, temperature, concentration, and time on material response is studied both to better understand the mechanism of change and to evaluate the use of accelerated test procedures.

ASSESSMENT OF SYNTHETIC MEMBRANE SUCCESSES AND FAILURES AT WASTE STORAGE AND DISPOSAL SITES

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ABSTRACT

Data provided by five vendors, from 27 lined facilities, were analyzed to determine the factors which contributed to success or failure of the liner at each of those facilities. The sites represented a wide variety of wastes handled, liner types, geographic locations, facility ages, facility sizes, etc. Based on the definitions used in this study, the 27 facilities selected by the vendors had a total of 12 "failures" at 10 sites. At four or five of these sites groundwater contamination apparently resulted from the failures. Some of the contributing factors, if not causes, for the failures include the following:

- Failure to control operations (at an operating site) so as to safeguard the liner;
- Poor (or inadequate) design work;
- Failure to use an independent, qualified design engineer;
- Poor (or inadequate) installation work;
- Poor or inadequate communication and cooperation between companies working on an installation job;
- The use of untrained and/or poorly supervised installers;
- Failure to conduct (or adequately conduct) waste-liner compatibility tests;
- Adverse weather conditions during installation;
- Use of old dump site, with contaminated soil, as site for lined facility;
- Selection of companies (for liner job) by processes that did not help ensure that good materials and workmanship would result;
- Selection of liner material by process not involving detailed bid specifications (prepared by design engineer, not liner manufacturer);
- Facility age (more failures were associated with the older sites).

Two main elements of membrane liner success are: (1) a proper philosophical and conceptual approach; and (2) the extensive use of quality assurance programs in all facets and stages of a facility's operation. Other factors noted as contributing to success included:

- Overdesign of a system;
- A knowledgeable customer;
- Bidding to specifications;
- Selection of qualified companies;
- Cooperation amongst companies on liner job;
- Conducting waste-liner compatibility tests;
- Simplicity of design; and
- Good weather.

RELATIONSHIP OF LABORATORY AND FIELD DETERMINED HYDRAULIC CONDUCTIVITY (K) IN COMPACTED CLAY SOILS

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ABSTRACT

Details of the field scale clay liner research facility and liner installation are given. The facility consists of a 30 x 75' bridge-like platform. It is instrumented with a grid of underdrains, gypsum blocks, infiltration rings, sites to measure swelling, density probe access tubes, inflow-outflow recorders, and evaporation pans. The clay is from the B-horizon of a Typic Hapludult (USDA Classification), or type CL (USCS Classification) soil, with a laboratory-determined hydraulic conductivity $K = 1 \times 10^{-8}$ cm/sec. It was compacted to 90% Standard Proctor using a crawler tractor pulling a sheepsfoot roller. The clay liner was placed in three lifts of 6" uncompacted thickness each. To facilitate quality control measurements with the nuclear density meter, the surface of each lift was smoothed with a smooth-face steel wheel compactor. Following the soil density readings, the lift surface was scarified by tracking with a bulldozer before the material for the next lift was applied.

Instrumentation to monitor moisture movement in the field facility was based on a priori prototype studies conducted in 10-feet diameter caissons under roof. These prototype studies have underscored the importance of adequate sealing of interfaces and elimination of liner perforations such as for example, wells or moisture-density access tubes. Prototype data suggest considerable variability from point to point. Consequently, a regular grid was imposed on the surface of the facility and measurements are being made at the specific times and locations. Using geostatistical techniques, areal distribution of infiltration and possible outcomes for differing infiltration or density scenarios can be projected.

A Standard Proctor mould was scaled-up using one-half of a 40-gallon steel drum. After soil was compacted into this mould, a 12" diameter ring was placed, which was filled with water to measure infiltration rate; the surface outside the ring was also flooded to provide the double-ring infiltrometer. The ring is the same size as will be placed at specified locations in the field facility. Hydraulic conductivity values obtained from the scaled-up mould will be compared with the distribution of values obtained both from the field facility and laboratory determinations.

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CRITICAL REVIEW AND SUMMARY OF GAS AND LEACHATE PRODUCTION FROM MUNICIPAL SOLID WASTE LANDFILLS

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ABSTRACT

The purpose of this project was to provide a review and critique of literature and the status of technology pertaining to the formation, characterization and treatment of municipal solid waste (MSW) landfill gas and leachate. This objective was accommodated by an initial recognition of the various factors influencing the applicability and efficacy of various landfill disposal options and/or leachate and gas management strategies. Therefore, the review is organized to reflect bench-, pilot- and full-scale evaluations; leachates have been categorized according to low, medium and high strengths; and product gases have been differentiated on the basis of constituents and medium or high energy content.

Commencing with a brief historical perspective focusing on the problems associated with migration of MSW leachate and gas from landfill disposal sites, the review proceeds with a characterization of MSW gas and leachate quality produced throughout the phases of landfill stabilization and the various leachate and gas treatment methods. Accordingly, leachate treatment by external aerobic and anaerobic biological methods and by in situ anaerobic degradation with leachate management are compared to physical-chemical leachate treatment by chemical oxidation, precipitation, coagulation, disinfection, absorption, ion exchange and reverse osmosis. In addition, methods of landfill gas collection and treatment by sorption, refrigeration, and membrane separation are presented together with the implications of ultimate disposal of treated leachates, including an understanding of leachate/soil interactions.

Biological treatment is shown to be superior to physical-chemical methods for raw leachate; aerobic and anaerobic treatment methods were generally capable of removing 85 to 99 percent of leachate BOD_5 and COD as well as most heavy metals. The physical-chemical processes are shown to be better suited for polishing of biological treatment effluents and for the removal of refractory organics. Leachate management through recycle is also demonstrated to possess a high potential for process stability, although further effort is needed to determine application criteria on a large scale. Process loadings, operational constraints, and degree of variance and uncertainty in reported data and their interpretation are presented to indicate that past efforts have not resolved all the technical challenges associated with gas and leachate management at landfill disposal sites, and that additional investigations are required.

HYDRAULIC CONDUCTIVITY OF TWO PROTOTYPE CLAY LINERS

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ABSTRACT

Two prototype compacted clay liners were constructed at a site near Austin, Texas, using clays of high and low plasticity. The liners were 15 cm (6 inches) thick and covered an area of approximately 6 meters by 6 meters. The clays were compacted to 100% of standard Proctor density at a water content slightly wet of optimum using a sheepsfoot roller. Small dikes were constructed around the perimeter of the test area, and water was ponded on the liners. An underdrain system was installed to collect liquid that flowed through the liner and to measure the quantity of seepage. The underdrain consisted of an impermeable geomembrane, freely draining filter fabric, washed gravel, and perforated collection pipes. The collected liquid flowed by gravity to a pit, where the quantity of flow was measured and was used to calculate hydraulic conductivity.

The actual hydraulic conductivity of the two liners was 4×10^{-6} cm/sec for the clay of high plasticity and 9×10^{-6} cm/sec for the clay of low plasticity. A variety of laboratory permeability tests were performed. The tests included permeability measurements on laboratory-compacted specimens of clay, on samples of the actual clay liners obtained with a thin-walled sampling tube, and on hand-carved blocks of clay obtained from the actual liners. Field permeability tests were also performed using single- and double-ring infiltrometers.

The various laboratory and field permeability tests showed that: (1) essentially all of the laboratory tests, even on undisturbed samples, produced a measured hydraulic conductivity that was one thousand times less than the actual hydraulic conductivity, and (2) field permeability tests yielded an average hydraulic conductivity that was close to the actual hydraulic conductivity.

The findings of this study raise important questions about whether laboratory permeability tests on compacted clay are relevant to field problems and reinforce previous suggestions that compacted clay liners may contain numerous hydraulic defects such as fissures, slickensides, zones of poor bonding between clods of clay, and zones of relatively poor compaction. The desirability of field permeability tests is evident from the results reported. Permeability tests on laboratory samples that were tested at different effective stresses suggest that clay liners subjected to significant overburden pressure may retain their hydraulic integrity much better than clay liners that have little or no overburden pressure.

CLAY-CHEMICAL COMPATIBILITY AND PERMEABILITY TESTING: A REVIEW

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ABSTRACT

It has been recognized for many years that certain chemicals can affect the permeability of clay soils. Several EPA-sponsored studies of the effects of selected organic and inorganic chemicals on clay soils have demonstrated the potential of some chemicals to increase clay permeability. Additionally, recent evidence from a hazardous waste disposal site in the United States suggests that the presence of an organic solvent, which formed a separate, denser than water phase in a clay-lined surface impoundment, contributed to the failure of the liner.

Although several researchers have suggested explanations for the observed chemically-induced changes in permeability, our review of the state-of-the-knowledge in this area concludes that current theories are inadequate to predict the extent to which complex chemical mixtures may affect clay soil permeability. Qualitative predictions can be made of the effects of certain concentration chemical solutions and nearly-pure solvents on selected clay soils. Previous reviews have identified mechanisms that could account for observed changes in permeability; use of this information to interpret test results and to predict chemical effects on clay permeability are complicated by the possibility that two or more of the processes may operate simultaneously.

The issues surrounding measurement and interpretation of permeability and chemically-induced changes in permeability are complex and EPA is sponsoring a number of studies on these topics. In January 1984, a 2-day workshop was convened to exchange research experience, discuss current findings, and to coordinate on-going and future work. Among the principle topics discussed were the following:

- Need for standard procedures
- Importance of sample preparation
- Effect of conditions (saturation, hydraulic gradient, etc.) on tests results
- Standard permeant solutions
- Chemistry of inflow and outflow solutions and soil sample
- Criteria for concluding compatibility tests without missing significant changes
- Criteria for appropriate test devices
- High priority chemicals for further testing

At least three different types of test devices (fixed-wall, flexible wall, and consolidation cells) are being used to measure clay soil permeability in the laboratory and a variety of different sample preparation methods and test procedures have been employed. The different approaches to measuring the effects of chemicals on clay permeability and the sensitivity of the tests

to a large number of variables has lead to test results that are often contradictory and difficult to interpret.

There is a divergence of opinion regarding the most appropriate laboratory test device. The large increases in permeability observed in tests using fixed-wall apparatus with certain organic liquids are attributed by some researchers to side-wall flow resulting from shrinkage of the clay sample on exposure to the chemicals. Although the lateral confining pressure in flexible wall cells prevents side-wall leakage, it may also prevent the development of shrinkage cracks that could be caused by the chemical displacement of water from the soil. The pressure applied to the surface of a sample in a consolidation cell may reduce leakage by forcing the sample into contact with the side-wall. There is some question about overburden pressures for use in testing clay liner material in consolidation cells.

Because of the effect of soil fabric on permeability and the difference in soil fabric between laboratory-compacted and field-compacted samples, much attention is being given to appropriate sample preparation for laboratory testing and to procedures for conducting tests in the field.

This review of the status and results of current research on clay-chemical compatibility and permeability testing was conducted as part of the production of a Technical Resource Document on design, construction, and evaluation of clay liners for waste management facilities.

CLAY LINER TRANSIT TIME PREDICTION METHODS AND EFFECTS OF LEACHATE COMPOSITION ON LINER INFILTRATION

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ABSTRACT

In order to evaluate an existing clay-lined facility or to design a new facility, two approaches to defining the transit time of leachates moving through a clay liner are recognized. The first approach is based on the movement of water or the saturation front in the liner. An alternative approach is based on the movement of leachate chemicals. For this project, the available prediction methods for transit time were evaluated for their applicability to either approach. The initial degree of liner saturation (water content), as well as the definition of transit time, determine the applicability of a given method.

Five analytical methods, as well as the numerical solution of the governing flow and solute transport equations, were reviewed. Three of the analytical methods solve both the solute transport and water flow equations and are based on the assumption that the liner material is completely saturated. Because the difference between molding water content and saturation are often small in compacted clays, these methods may be applied to partially saturated clays without major error. The other two methods describe the infiltration dynamics in unsaturated materials but do not take into consideration solute transport.

Limitations in applying these methods in clay liner design arise from the data requirements and assumptions that must be made. For example, all the transit time prediction methods assume a homogeneous liner and do not consider the effect of leachate composition directly. Numerical methods make fewer assumptions, but are expensive, and the accuracy achievable depends upon the quality of available data. Numerical solutions also require specific expertise in available models.

Since clay liners are compacted wet of optimum (i.e., at a moisture content slightly greater than the moisture content at which maximum density can be achieved) their condition differs from the completely saturated (zero air void) clay systems that are used typically to determine clay permeabilities. Thus, test data from unsaturated compacted clay soils are needed to assess the reliability of computer models for predicting pollutant transport in unsaturated clay soils.

Clay-lined hazardous waste landfills accept chemical wastes which produce leachates containing many chemical components. These chemicals in the leachate may contact and migrate into the clay liner where they could affect clay structure and ability to

control movements of fluids (permeability). Because of this, a companion investigation was performed to determine if leachate composition can significantly change the rate of liquid movement during infiltration into unsaturated compacted clay liners.

Three native clay soil samples, compacted in plexiglass cylinders (4.25" OD and 5" long) were exposed to test liquids under a constant 1 foot head. The solutions tested were 0.01N $CaSO_4$ (control), sulfuric acid (1 percent), sodium lauryl sulfate (1 percent), and methanol (8 percent). After several months of exposure, the soil columns were extruded and sliced. Analyses were performed to determine moisture content and chemical components at various depths in the soil column. The rate of liquid movement into the clay was calculated from these analyses and any liquid flow out of the clay was measured.

EFFECTIVE POROSITY OF COMPACTED CLAY SOILS PERMEATED WITH ORGANIC CHEMICALS

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ABSTRACT

Total porosity has commonly been considered as the available pathway for flow of impounded liquids through compacted clay soil liners. The validity of this assumption was investigated by estimating the fraction of active total pore space in several soils for conduction of permeant liquids. The morphology of three compacted sandy clay loam soils permeated with three liquids, e.g., water, acetone, and xylene wastes, is discussed and illustrated. The soils are differentiated by their dominant clay mineralogy: kaolinitic, bentonitic, or micaceous. Total porosity was obtained from bulk density calculations Effective porosity was estimated from microscopic observations, which included estimates from (1) Planimeter measurements on micrographs from scanning electron microscopy (Backscatter mode), (2) Ribbon-traverse counts on thin sections, observed under epifluorescent, and (3) Petrographic light- microscope observation. In some cases, less than 10% of the total porosity was effective in transmitting the liquids. Photomicrographs showing examples of observations from the microscopic techniques are presented.

The macromorphology of the soils after permeation is illustrated in photographs of excavated field test cells where compacted clays were exposed to industrial solvent paint wastes. Two dyes, one visually detectable and the other fluorescent under ultraviolet light, were added to each waste liquid at the beginning of the study. At the time of excavation of the simulated liners, visual observations were made of the fluid flow path through the compacted clay layers using both the visible dye and pigments contained within the waste liquids. The dominant macroscopic flow paths through the kaolinitic and micaceous xylene-waste permeated soils were planes that were believed to have been induced by the compaction process. Microscopic studies supported the visual observation. Effective porosity predominated in pores greater than 125 micrometers average diameter, which comprised less than 1% of the total soil volume for these soils with xylene-based waste.

The bentonitic soil demonstrated a higher effective porosity in the presence of xylene but was still less than 3% of the total soil volume. Visual observations showed more numerous soil surfaces dyed within bentonitic soil than those of the other two mineralogies, suggesting a greater effective porosity.

Less than 2% of the total soil

volume was effective in transmitting acetone or water in micaceous and kaolinitic soils. Nearly 5% of the total soil volume was effective in transporting both water and acetone in the bentonitic soil. Soils permeated with acetone-waste in field cell studies contained essentially the same degree of effective porosity as the same soils in laboratory permeameters.

Total porosity was measured from portions of each soil taken near the surface of the compacted bulk, which has been shown by other investigators to be the most dense portion of the soil and, thus, the most restrictive to liquid flow. Total porosity in each of the three soil studies was as follows: bentonitic soil - 14.5%; kaolinitic soil - 11.6%; and micaceous soil - 13.2%. The compacted clay soil pore space that was active in allowing liquid to flow through the soil was demonstrated, by the available data, to be substantially less than the pore space calculated as the total porosity.

INFLUENCE OF CONCENTRATIONS OF ORGANIC CHEMICALS ON THE COLLOIDAL STRUCTURE AND HYDRAULIC CONDUCTIVITY OF CLAY SOILS

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ABSTRACT

The impact of concentrated organic chemicals on the permeability of compacted clay landfill and pond liners is now reasonably well documented. The response of clays to concentrations of nonpolar compounds which do not exceed their water solubility levels have been shown to be similar to that of water. As the concentration of polar compounds increase, their impact on the permeability of clay liners appears to shift from that of water to that of the specific organic liquid.

The present study was undertaken to determine the concentrations at which this shift in behavior occurs and to evaluate the mechanisms that regulate the shift. Three different clays were suspended in a series of aqueous dilutions of acetone and ethanol in hydrometer jars. The percent apparent clay was used as an index of the flocculation of the clays. In all cases, the clays remained dispersed in water,but as the concentration of the organics increased, a concentration was reached above which the clays quickly flocculated and settled to the bottom of the cylinders. Flocculation occurred at dielectric constants in the range of 30 to 50 for all the systems. Conductivity measurements made in fixed wall permeameters with similar dilutions on the clays indicated that marked increases occurred at concentrations above which the clay flocculated.

The results indicate that when organics are sufficiently concentrated, the clay will flocculate and result in shrinking, cracking and large increases in hydraulic conductivity. Flocculation tests or dielectric constants may possibly be used as a tool to predict the influence of a given solution on the behavior of clay liners. However, more accurate indications of clay liner/leachate compatibility may be possible by using other parameters which better characterize the interaction of clays and solutions.

LABORATORY DETERMINATION OF DIELECTRIC CONSTANT AND SURFACE TENSION AS MEASURES OF LEACHATE/LINER COMPATIBILITY

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ABSTRACT

Research on hydraulic conductivity of compacted clay soils using various permeant liquids has shown the trend that as liquids of lower dielectric constant are employed, the soil hydraulic conductivity increases. These observations suggested that there may be a reliable relationship between the clay soil's performance as a landfill liner and readily measured properties of leachate liquid which the liner must contain. Both industry and agencies would benefit from the confirmation of a rapid and easy test to measure and/or predict liner/leachate compatibility.

This report compiles, from published technical reports and laboratory studies, dielectric constant and surface tension measurement data for several common organic solvents and selected binary mixtures. Dielectric constant was examined both because published reports cited this key parameter in relation to clay soil permeability and a commercial instrument was available to measure this property of various liquids.

Surface tension was measured because of the potential relationship to pore entry pressure of non-aqueous materials in mineral matrices, such as clay soils, and the relatively easy laboratory measurement.

Data will be presented which demonstrate the laboratory performance of a dielectric constant meter and surface tensiometer using various solvents and mixtures. Operational precautions and instrumental peculiarities will be discussed. Data generated will be compared with published reports of the hydraulic conductivity of compacted clay soils using liquids the same as or similar to those examined in this study.

QUICK INDICATOR TESTS TO CHARACTERIZE BENTONITE TYPE

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ABSTRACT

Bentonite-soil mixtures are a primary element of slurry wall waste containment barriers. With several different bentonite products commercially available, selection and specification of a particular bentonite type is an application-dependent decision. Cost and technical performance considerations are frequently applied only on a subjective basis. Commercial product descriptions generally categorize bentonites as ordinary (unaltered), physically altered, polymer protected or extended, or chemically resistant types. Currently there are no known quick field laboratory test protocols which allow a design engineer's quality control program to distinquish bentonite type or to evaluate in-service performance under chemical or leachate attack.

The purposes of this project are: 1) to identify a practical indicator test(s) to distinguish among bentonite types; 2) to develop a data base which characterizes various bentonite types in terms of chemical/leachate impact; and 3) to relate measured impact to in-service hydraulic conductivity of the soil-bentonite mixture.

Research to date has focused on bentonite type distinction and chemical/leachate impact characterization. Ten conventional laboratory classification and index tests employed in environmental science, industry, and geotechnical engineering were selected for study. Data characterizing the impact of three liquids on various test parameters were generated for ten of approximately 42 commercially available bentonite products.

Preliminary findings on the basis of limited data obtained to date are: (1) Tests designed to measure bentonite slurry gel strength, free swell potential, and various viscosity parameters appear to have strong potential for distinguishing among bentonite types; (2) Bentonite hydration in a 3:1 water:acetone solution (25% acetone) enhanced desirable engineering properties of bentonite relative to water hydration results in about 85 percent of the tests; (3) Bentonite hydration in a 1:3 water:acetone solution (75% acetone) degraded desirable bentonite properties relative to water hydration in about 69 percent of the tests; (4) For bentonites hydrated in water or acetone:water solutions, chemically resistant, ordinary (unaltered), and physically altered amine-treated bentonites were distinguishable from the other types tested.

Future research through 1985 will involve refinement of bentonite type classifications, different water:acetone leachate concentrations, evaluation of other potentially useful indicator tests, and bentonite characterization on the basis of chemical or leachate impact after full water hydration.

ESTIMATING TRANSIT TIMES OF NONINTERACTING POLLUTANTS THROUGH COMPACTED SOIL MATERIALS

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ABSTRACT

Pollutant travel times through compacted soil materials cannot be accurately predicted from the permeability (saturated hydraulic conductivity) alone. Travel time is also dependent on the effective porosity of the material; i.e., the portion of the total porosity that contributes significantly to fluid flow. Once permeability and effective porosity are determined for a selected material, travel times for noninteracting pollutants through specified thicknesses of compacted material at specified hydraulic gradients can easily be predicted. This paper presents a straightforward method of determining the effective porosity of compacted soil materials and compares measured and predicted solute breakthrough times for three compacted soil materials.

The determination of effective porosity is based upon the total porosity and the spread on a log scale in the pore sizes of a compacted sample. Once the total porosity and pore size distribution information are obtained for a particular sample, the effective porosity can be determined by using a graphical solution.

Pollutant travel time, T, through a compacted soil (i.e., when pollutant first appears at the bottom of the liner) can be predicted by T=EL/KI; where E is effective porosity, L is thickness of compacted sample, K is sample permeability, and I is hydraulic gradient. Chloride travel times through compacted samples of glacial till, loess, and paleosol materials were measured as less than 208 minutes (min.), between 15 and 30 min., and between 250 and 380 min., respectively. Predictions of noninteracting solute breakthrough for the glacial till, loess, and paleosol materials were 227 min., 32 min., and 270 min., respectively. Thus, the predictions are reasonably close to measurements. Further experiments using thicker soil samples are under way.

INTRODUCTION AND PURPOSE

Compacted clay is frequently used to line hazardous-waste-disposal landfills. The purpose of the compacted clay liner is to restrict the movement of hazardous liquid materials out of the landfill. Currently, soil permeability (saturated hydraulic conductivity) is the most common measurement used to estimate the potential effectiveness of the liner material (USEPA, 1978). Soil permeability, K ($m^3/m^2/s$), is defined from Darcy's equation:

$$K = J/I \qquad (1)$$

where J is the fluid flux density ($m^3/m^2/s$) and I is the hydraulic gradient (m/m). Soil permeability has dimensions of volume per unit area of liner per unit time; thus, it is a bulk parameter related to the areal average fluid flow in the liner. But knowledge of average fluid flow does not allow the prediction of pollutant breakthrough or travel times. To predict pollutant travel times through clay liners, the liner effective porosity must be known.

Effective porosity, E, has been described as that portion of the total liner porosity that contributes significantly to fluid flow. Effective porosities are less than total porosities because some of the pore space is discontinuous (dead end) and some of the pore space is so narrow that fluids in these spaces are essentially immobile. If the fluid flux density through a clay liner can be determined by using

Darcy's equation, then the mean effective fluid velocity, $\bar{V}_E$, can be described by:

$$\bar{V}_E = KI/E \quad (2)$$

If the hydraulic gradient is constant and liner K and E are both known, then the average pollutant travel distance per unit time can be estimated. Thus, the time of first breakthrough, T, of a noninteracting pollutant can be predicted by the equation:

$$T = EL/KI \quad (3)$$

where L is the liner thickness (m).

The Environmental Protection Agency, EPA, requires that a disposal unit liner prevent migration during the active life of a unit (USEPA, 1982). The active life of a unit containing noninteracting pollutants can be estimated by using Eq. (3). Knowledge of the liner permeability is not sufficient information to accurately estimate the length of time of liner effectiveness. When a disposal unit is constructed with known liner thickness, L, and is designed for a known hydraulic gradient, I, measurement of both permeability and effective porosity are required to estimate the active life of the unit with Eq. (3). Although methods of measuring permeability of compacted clay materials have received much attention in the literature (Olson and Daniel, 1981), determination of effective porosity has received little attention.

The purpose of this paper is to present a method for estimating the effective porosity of compacted clay materials. The estimates of effective porosity are used in Eq. (3) to predict transit times of noninteracting solutes through samples of compacted materials. The predictions of transit time are compared with measured chloride breakthrough times.

APPROACH

Theory

To estimate the travel time of noninteracting pollutants through a liner by Eq. (3), the liner's effective porosity must be determined. We earlier described effective porosity as that portion of total porosity that contributes significantly to fluid flow. We now define "significance" by defining effective porosity as that portion of total porosity that conducts fluid faster than the average pore-water velocity. Large pores conduct fluid faster than do small pores. Therefore, in a porous medium that exhibits a range of pore sizes, a range of pore-water velocities will exist as a fluid passes through. Higher-than-average pore-water velocities in the larger pores will be responsible for the first appearance of pollutants, whereas lower-than-average pore-water velocities associated with smaller pores will not be responsible for first appearance of pollutants. Thus, the determination of a liner's porosity versus pore-water velocity distribution is crucial to the evaluation of the liner's effective porosity.

The pore-water velocity distribution can be calculated from the unsaturated permeability, $K(\theta)$, as a function of fluid-filled porosity, θ. On the basis of theory developed by Mualem (1976), Van Genuchten (1980) showed that the unsaturated permeability could be predicted from the total porosity, the residual porosity (i.e., the value of porosity when water films lose effective continuity), and a measure of the uniformity of pore sizes by the equation:

$$K(\theta) = K\left[\frac{\theta-\theta_r}{\theta_s-\theta_r}\right]^{\frac{1}{2}}$$

$$\left[1-\left(1-\left(\frac{\theta-\theta_r}{\theta_s-\theta_r}\right)^{1/m}\right)^m\right]^2 \quad (4)$$

where θ_s is the total porosity, θ_r is the residual porosity, and $m = [1-(1/n)]$ for $0<m<1$; n is an empirical parameter that describes the uniformity of pore sizes. θ_s, θ_r and n can be derived from a cumulative porosity curve.

Cumulative porosity curves may be obtained by either soil-water desorption techniques or mercury-intrusion porosimetry. Most engineers prefer the latter method because of its speed. Both approaches are based on the capillary-rise equation, which, for mercury porosimetry, may be formulated:

$$r = (1/P)(2\gamma \cos\partial) \quad (5)$$

where r = pore radius (m), P = pressure (Pa), γ = surface tension of mercury (N/m), and ∂ = wetting angle of mercury (140°). To obtain θ_s, θ_r, and n from mercury-intrusion curves, we adapted an equation originally developed by Van Genuchten (1980) for soil-water desorption curves:

$$\theta = \theta_r + (\theta_s-\theta_r)$$

$$[1 + (\alpha/r)^n]^{(1/n)-1} \quad (6)$$

where r is the pore radius in Eq. (5) and α is an empirical parameter. To use Eq. (6) to model mercury-intrusion curves, we define θ_s as equal to total porosity of the freeze-dried samples and transform the mercury-intrusion cumulative porosity to a form similar to the soil-water characteristic curve as follows: $\theta = \theta_s$ - mercury-filled porosity. Once the cumulative porosity curves are transformed, the θ_r, n, and α parameters(Eq. 6) can be determined by using nonlinear regression techniques.

When nonlinear regression techniques are not accessible, a first approximation of the θ_r and n parameters can be determined from a graphical display of the transformed data. The residual porosity, θ_r, is approximated as the value of θ that the cumulative porosity curve asymptotically approaches as limiting pore size decreases. Mercury-intrusion cumulative porosity curves for compacted soil samples generally have θ_r -values approximately equal to their non-mercury-filled porosities after intrusion to 414 MPa (60,000 psi). For soil materials considered for landfill liners, θ_r is generally less than 0.1. The n-parameter is estimated as follows: The cumulative porosity data are plotted as θ vs. log r; next, the absolute value of the slope, S, of the cumulative porosity data is calculated at a value of approximately halfway between θ_r and θ_s. The n-parameter is then determined by using the equation:

$$n = \begin{cases} \exp\,(0.8\ Sp) & ;\ (0<Sp\leq 1) \\ 1/\left(\dfrac{0.5755}{Sp} - \dfrac{0.1}{Sp^2} - \dfrac{0.025}{Sp^3}\right); & (Sp>1) \end{cases} \qquad (7)$$

where Sp is equal to $S/(\theta_s - \theta_r)$.

Once θ_r and n are estimated from the measured cumulative porosity data, the sample effective porosity can be determined from Figure 1 (for a detailed explanation, see Horton, 1985). With an estimate of the effective porosity, the time required for the first appearance of noninteracting pollutants at the bottom of a clay liner can be predicted by using Eq. (3).

Soils

We collected subsurface materials from three soils developed in major Iowa parent materials: till, loess, and paleosol. The till-derived soil was Nicollet composed of sandy clay loam materials (Unified Classification: CL; AASHTO classification: A-4). The loess-derived soil was Fayette composed

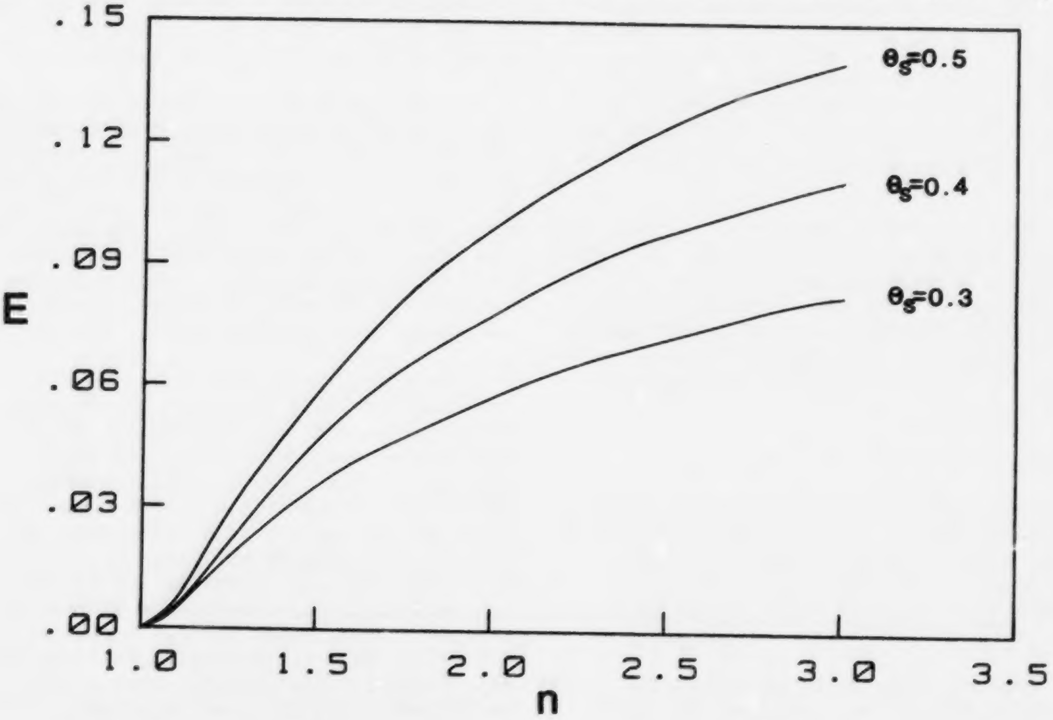

Figure 1. Influence of n on effective porosity (E) at various values of θ_s (θ_r = 0.05).

TABLE 1 PARTICLE SIZE DISTRIBUTION, OPTIMUM MOISTURE CONTENT AND BULK DENSITY OF THE COMPACTED SOIL MATERIALS

Soil Material	Clay (<2μm)	Silt (2-50μm)	Sand (50-2000μm)	Optimum moisture content	Bulk density (g/cm^3)
	%	%	%	%	
Nicollet	21	25	54	14	1.84
Fayette	35	57	8	18	1.67
Clarinda	44	35	21	27	1.45

of silty clay loam materials (Unified classification: CL; AASHTO classification: A-7). Clarinda was an exhumed paleosol composed of clay materials (Unified classification; CH; AASHTO classification: A-7). These soils were chosen to be representative of the kind of materials likely to be used in construction of landfills in Iowa.

The three soil materials were distinguished by their particle size distribution (Table 1). Nicollet materials had the least clay and the most sand, whereas Clarinda materials had the most clay and an intermediate sand content. Fayette materials, derived from loess, were highest in silt.

Soil materials were collected by excavation, air-dried, and ground to pass a 2-mm sieve. A standard moisture-density relationship for each soil material was determined according to ASTM standard test D-698-78 Method A. The optimum moisture contents and bulk densities are reported in Table 1.

Porosity

Samples were prepared by compacting soil materials at moisture contents 1 to 2% higher than optimum. Compacted samples were broken into fragments <3 cm^3. The fragments were freeze-dried according to the method of Zimmie and Almaleh (1976).

Cumulative porosity of triplicate fragments of each soil material was measured by using a Quantachrome SP-200 mercury porosimeter in four intrusion steps: 0 to 0.1 MPa, 0.1 to 8.3 MPa, 8.3 to 41.4 MPa, and 41.4 to 414 MPa (60,000 psi). Cumulative porosities were plotted as cumulative volume vs. log pore radius.

Chloride Breakthrough Measurements

From the moisture-density relationship, a value one or two percentage points above optimum moisture was chosen at which to moisten the soil. The soil was moulded in a 6.35-cm diameter by 2.54-cm high cylinder at the corresponding bulk density.

Each compacted soil sample was placed in a 7.62-cm diameter acrylic permeameter. Paraffin, kept at 60°C, was poured between the wall of the permeameter and the sample in 0.5-cm increments until the top of the compacted sample was reached; the paraffin was allowed to solidify between increments. In this manner, any gaps formed when the paraffin cooled were filled by the next increment, thus minimizing wall effects.

De-aired, saturated $CaSO_4$ solution (adjusted to 0.06% formaldehyde, to control microbial growth) was introduced into the permeameter and pressurized from an air-pressure source. The air was separated from the solution by means of a rubber membrane within the solution container. Hydraulic gradients in the range of 100-200 were used; because of the relatively small sample height, this required a hydraulic head of approximately 2.5-5.0 m H_2O (3.5 - 7.5 psi). Once the hydraulic gradient was established, a minimum of 5 pore-volumes was passed so as to obtain a constant flux from which to determine the sample permeability.
A de-aired, 0.1N $CaCl_2$ solution with 0.016%

Acid Fuchsin, a red dye, and 0.06% formaldehyde was used as the tracer solution for the solute breakthrough studies. The dye gave a visual test for any wall effects. The tracer was exchanged for the $CaSO_4$ solution and allowed to leach through the soil sample under the same hydraulic gradients. The leachate was collected in equal-volume increments with a fraction collector; a minimum of 3 pore-volumes was collected. The leachate was analyzed for chloride concentration with an automatic titrator.

PROBLEMS ENCOUNTERED

The problem of greatest concern was whether soil porosity was altered during sample drying preceding mercury porosimetry measurements. Our method of determining effective porosity depends on both the cumulative pore size distribution and the total porosity. Yet, to determine pore size distribution by mercury porosimetry, all water must be removed from the sample. Some of the total porosity in the samples was lost when water was removed by freeze drying (Table 2).

TABLE 2 TOTAL POROSITIES OF COMPACTED SOIL MATERIALS BEFORE AND AFTER DRYING

Soil Materials	Before drying	After drying
Nicollet	0.29	0.28
Fayette	0.40	0.40
Clarinda	0.46	0.40

As Table 2 shows, some loss of porosity occurred in the Nicollet and Clarinda samples during freeze drying. The greatest decrease in porosity was with Clarinda soil materials, the materials highest in clay content. Total porosity in Fayette soil materials was not diminished by freeze drying. The significance of the losses in total porosity to calculations of effective porosity does not seem to be large.

RESULTS

The function used to describe the cumulative porosity curve (Eq. 6) has two empirically determined parameters, α and n. Only the n-parameter is required to describe the unsaturated permeability. Figure 2 presents a family of curves over a range in n from 1.1 to 5. The curves show that the spread on a log scale of pore sizes decreases as n increases. Thus, it becomes clear that the theory provided for calculating unsaturated permeability considers the spread in pore size distribution as the predominant factor.

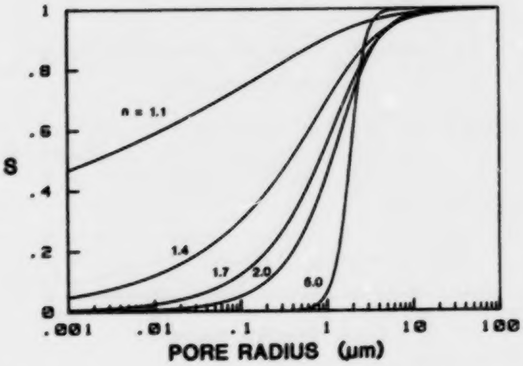

Figure 2. Theoretical cumulative porosity curves as influenced by the n-parameter (α= 1.0, S= θ/θ_s).

Figures 3, 4, and 5 show the cumulative porosities for Nicollet, Fayette, and Clarinda soil samples, respectively. The discrete, plotted symbols represent values determined by mercury porosimetry. The solid curves represent the least-squares fit of Eq. (6) to the measured data. In all cases, the model does a reasonable job of matching the measured data. The α, n, θ_r and θ_s parameters for each model are presented in Table 3.

Equation (4) describes the unsaturated permeability of soil once the n-parameter is known. Figure 6 presents a family of curves displaying the influence of the n-parameter on unsaturated permeability. As n increases, the relative permeability tends to decrease more gradually as the soil begins to desaturate. This indicates that, the larger the n-parameter, the larger is the fraction of porosity with relative importance for conducting fluids. As n increases, one can expect the effective porosity to represent a larger portion of the total liner porosity.

Figure 1 presents a family of curves, over a range in θ_s from 0.3 to 0.5, dis-

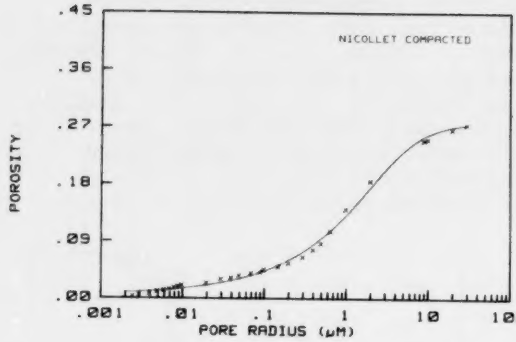

Figure 3. Measured and curvefitted cumulative porosity for compacted Nicollet soil samples.

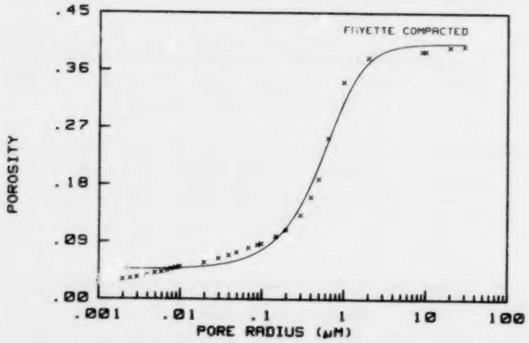

Figure 4. Measured and curvefitted cumulative porosity for compacted Fayette soil samples.

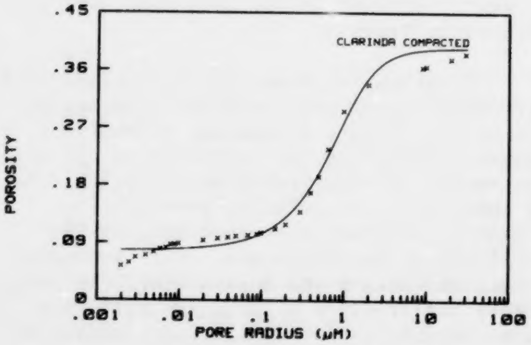

Figure 5. Measured and curvefitted cumulative porosity for compacted Clarinda soil samples.

TABLE 3 PARAMETERS USED WITH EQUATION (6) TO DESCRIBE THE CUMULATIVE POROSITY OF THE SOIL MATERIALS.

Soil Materials	α^*	n^*	θ_r^*	$\theta_s^{\#}$
Nicollet	4.08	1.52	0.00	0.28
Fayette	0.88	2.11	0.05	0.40
Clarinda	1.22	1.98	0.08	0.40

*/ α, n, and θ_r were determined by least-squares nonlinear regression of mercury intrusion porosimetry data and Eq. (6).

#/ θ_s was specified as the total porosity of the soil samples after freeze drying.

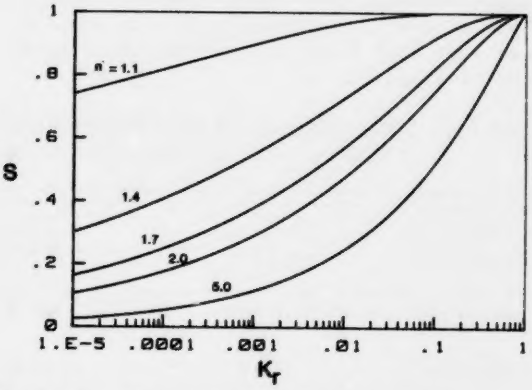

Figure 6. Theoretical relative permeability (K_r) curves as influenced by the n-parameter ($S = \theta/\theta_s$).

playing the influence of n on the effective porosity when θ_r is constant at 0.05. If similar sets of curves were drawn for θ_r = 0.0 and θ_r = 0.10, only small differences from Fig. 1 would be observed; i.e., the maximum difference in E would be 0.007. Thus, Fig. 1 can be used to obtain estimates of effective porosity of compacted clay liners with θ_s between 0.3 and 0.5, θ_r between 0.0 and 0.1, and n between 0.1 and 3.0.

The method developed in this paper for estimating the effective porosity of compacted clay liners depends only on parameters describing liner pore structure;

TABLE 4 MEASURED PERMEABILITIES AND TIME RANGES OF FIRST CHLORIDE BREAKTHROUGH AND PREDICTED EFFECTIVE POROSITIES AND NON-INTERACTING POLLUTANT BREAKTHROUGH TIMES FOR THE COMPACTED SOIL MATERIALS.

Soil Material	Permeability ($m^3/m^2/s$)	Measured Cl^- breakthrough time (min)	Effective porosity	Predicted breakthrough time (min)
Nicollet	5×10^{-10}	208	0.04	227
Fayette	2×10^{-8}	between 15 and 30 min	0.08	32
Clarinda	1×10^{-9}	between 250 and 380 min	0.08	272

i.e., θ_s, θ_r, and n-parameter. Cumulative porosity measurements provide the basis for determination of the appropriate parameters. By using the pore structure parameters presented in Table 3, effective porosities for the Nicollet, Fayette, and Clarinda soil materials are estimated from Fig. 1 as 0.04, 0.08, and 0.08, respectively. Once effective porosity is estimated, pore fluid velocity and noninteracting pollutant travel time can be calculated for a compacted liner with a known permeability, hydraulic gradient, and thickness.

Table 4 presents measured values of permeability and times of first chloride breakthrough, estimated effective porosities, and predicted times of first breakthrough of noninteracting pollutants for compacted Nicollet, Fayette, and Clarinda samples. Nicollet and Clarinda samples both had permeabilities equal to or less than the preferred value of 1×10^{-9} $m^3/m^2/s$. The breakthrough measurements were reported for time ranges because effluent volume increments were collected for analysis of tracer concentrations. The estimated effective porosities were between 15 and 20% of the total porosities of the compacted samples. The predicted noninteracting pollutant breakthrough times agreed quite well with the measured chloride breakthrough times. Predicted breakthrough times for the Nicollet and Fayette samples were slightly later than the measured breakthrough time ranges. The predicted breakthrough time for the Clarinda sample was within the measured breakthrough time range. Because the clay fractions of the compacted samples had a net negative charge and chloride is an anion, it is reasonable to assume that chloride would breakthrough the samples sooner than a noninteracting pollutant (i.e., tritiated water).

Future work will include breakthrough measurements using tritiated water as the tracer. The thickness of the compacted samples will be increased for further breakthrough studies. The pore size distributions of other compacted materials suitable for landfill liners will be further investigated. Perhaps evidence can be acquired to show that the n-parameter of compacted liner materials has a narrow range (i.e., 1.5 to 2.5) and, thus, indicate that the effective porosities represent a narrow percentage range of the liner total porosities (i.e., 15 to 25%). Such findings will enable easy, yet reasonable, estimates of the active lifetimes of hazardous-waste-disposal units.

EXAMPLE

The theory presented provides the basis for estimating effective porosity of compacted clay liners. To predict the travel times of noninteracting substances through the clay liner, the hydraulic gradient, liner permeability, liner thickness, and effective porosity are required. As an example of how the theory can be used, we will make such a prediction. The information that we assume is: liner thickness = 1 m, hydraulic gradient = 1.33, liner permeability = $1 \times 10^{-9} m^3/m^2/s$.

If we follow only the Darcy equation, Eq. (1), fluid flux density is 1.33×10^{-9} $m^3/m^2/s$. However, the fluid flux density does not represent the fluid velocity within the liner. We know that a portion of the total porosity is responsible for conducting fluid at velocity higher than the average and that the remaining portion of the porosity conducts fluids at velocities lower than the average value. What is important in pollutant transport is the portion of porosity that conducts fluid at higher velocities. To determine the average effective fluid velocity, we must first estimate the effective porosity.

Effective porosity is determined on the basis of theory previously presented. By using the mercury porosimetry data for the Clarinda materials presented in Fig. 3 and Eq. (7), the n-parameter is determined as 2.0, θ_r as 0.08, and θ_s as 0.40. The effective porosity then, is estimated to be 0.08 by using Fig. 1. From Eq. (3) the travel time for noninteracting substances through a Clarinda clay liner in this example is predicted to be 2 years. Therefore, under these conditions, we predict that after 2 years, noninteracting pollutants will reach the bottom of the clay liner. Although the concentration of the noninteracting pollutant may be low because of lateral mixing by diffusion, the initial fraction is predicted to appear after 2 years. In this example, if the product of liner permeability and hydraulic gradient (i.e., fluid flux density, J) is mistakenly used as the fluid velocity, the pollutant travel time is calculated to be 23.8 years. If the fluid velocity is mistakenly assumed to be equal in all soil pores, the noninteracting pollutant travel time is calculated as 9.5 years. Thus, travel time for noninteracting pollutants is significantly overestimated if effective porosity is not taken into account. The active lifetime of hazardous-waste-disposal units may be overestimated if liner effective porosity is not properly estimated and used in the prediction of pore-fluid velocity.

ACKNOWLEDGMENTS

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WASTE-LINER COMPATIBILITY STUDIES

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ABSTRACT

Three different studies were undertaken focusing on the compatibility of synthetic liners (principally flexible membrane liners [FML]) with hazardous wastes. Each of the three studies is described herein under its final report title. In Study I, qualitative and quantitative information on the chemical resistance of FML materials was collected from vendor and technical publications. This information was compiled into a computer data base comprising about 3,000 data fields on 23 liner materials and over 500 chemicals. Criteria for assessing this information on a common basis were developed, with separate criteria for olefinic and non-olefinic liner materials. On the basis of these criteria, normalized ratings of chemical resistance were developed for each chemical/ material pair for which there were data. The Study report contains these ratings (over 1,300 of them) as well as a listing of the raw data from which they were derived.

In Study II, a search was made for laboratory test methods that were, or could be, used to ascertain the compatibility of flexible membrane liners with hazardous wastes. Twenty-seven identified methods, from the U.S. and several foreign countries, were examined and compared. Each pertinent test is described in the Study report. The Study report also discusses the philosophical basis of these tests and the extent to which they can realistically be expected to reflect short- and long-term liner performance.

In Study III, a research effort was undertaken to evaluate routes to chemical/liner compatibility estimation. Proven estimation techniques might be used, for example, in prescreening FML materials to be tested for compatibility with a waste, or estimating the resistance of an existing FML to a new chemical in the waste. This study showed that one approach using solubility parameters can provide useful information in many situations. Solubility parameters were shown to be useful in three different ways, in solubility parameter maps, statistical correlations, and with the use of the exchange energy density parameter.

STUDY I - COMPATIBILITY OF FLEXIBLE MEMBRANE LINERS WITH CHEMICALS AND WASTES

INTRODUCTION

At present there are four primary and approximately ten secondary base polymeric materials from which flexible membrane liner (FML) sheeting is made. In lined RCRA facilities, the FML liners may be exposed to a variety of chemicals some of which will be known in advance, and some which will not have been identified or may be added later. The process of selecting the most appropriate liner material (for a new facility) or of checking to see if a modified waste stream (containing new chemicals, for example) will be compatible with the in-place liner can be a time-consuming and expensive undertaking. Selection of a liner material for a new RCRA site, including confirmatory compatibility tests, could take, for example, over six months and cost several thousand dollars.

Over the last several years chemical compatibility tests have been conducted by the liner industry, by those organizations which are contemplating installation of a waste impoundment facility, and by various government agencies. Although these groups have conducted a significant amount of testing, the use and/or comparison of their results has been difficult because there have been no standard procedures for testing or reporting the findings. Furthermore, the findings are scattered throughout the technical literature, government reports, vendors' brochures, and proprietary and unpublished literature.

In spite of the above limitations, there is a clear need to compile the available data so that maximum use can be made of past work. A compiled and evaluated data base on waste-FML compatibility may be of use if preliminary screening for FML selection and in determining compatibility for in-place FMLs to be exposed to new waste compositions. Such a data base would also become a valuable tool in evaluating different test protocols that have been used, in identifying major data gaps, and in the development of FML-chemical compatibility prediction schemes.

PURPOSE

The purpose of this study was to gather data on the compatibility of FML and various chemicals and waste liquids, and to analyze and report these data on a common basis. The intent of this compilation of chemical compatibility information is to assist liner material vendors and purchasers, and those involved in reviewing permit applications for lined facilities, in ensuring that - at least from a chemical compatibility standpoint - the most appropriate material is installed at a waste site.

APPROACH

The study was composed of three principal components:

(1) Collection and compilation of liner-waste compatibility information;
(2) Evaluation of information; and
(3) Development of a liner-waste compatibility matrix.

The information was collected from the open literature and from brochures and data sheets obtained from the liner industry. Additional data were sought from European investigators, but the yield was minimal. While the focus was on liners fabricated from synthetic polymers (FMLs), asphalt liners were also addressed. Both qualitative and quantitative compatibility data were collected.

The information was compiled using commercially available database software. The format allowed easy review of all available data on a given liner-chemical or liner-waste pair at once and the formulation of an overall judgment as to the compatibility of the liner material with the waste. The results were summarized in the form of a compatibility matrix, organized such that all chemicals within a given family of chemicals (e.g., aldehydes, ketones, acids) are reported together. This may facilitate extrapolation of the resistance information to liner-waste combinations for which no data are available.

Finally, all the information (test data and qualitative ratings) were tabulated and included as an appendix to the main report (Schwope et al., 1984). This will allow others (who may wish to independently evaluate compatibility) access to the original data.

RESULTS

The qualitative data, usually provided by FML vendors as FML-chemical compatibility charts, typically assign one of two or three ratings to each FML-chemical pair (for specified conditions, eg., temperature). Examples of compatibility ratings include the following:

Company A

S = Laboratory data or field experience indicate satisfactory service likely
U = Not recommended, unsatisfactory service likely

Company B

O = No effect
M = Moderate effect
S = Severe effect

Company C

S = Satisfactory
L = Limited application possible
U = Unsatisfactory

Company D

E = Less than 10% change in volume, ultimate elongation or tensile strength
G = Less than 30% change in volume
P = More than 30% change in volume

These ratings were all converted to a normalized basis using a rating ranging from "a" to "d" where "a" is the most resistant and "d" is the least resistant. The combinations "ab" and "bc" were used for a few cases where the vendor's description of the degree of compatibility appeared to be intermediate between our classification ratings. Ratings of "a", "ab" and "b" are taken as indicative of acceptable resistance or compatibility; ratings of "bc", "c" and "d" are taken as indicative of unacceptable resistance.

Table 1 provides, for example, the normalized ratings from three high density polyethylene (HDPE) vendors for seven chemicals. Note that there is essentially complete agreement among the vendors in this case. One would expect, perhaps, significant variability for a polymeric product that for specifc purposes, is produced with varying amounts of fillers and other compounding agents.

The evaluation and use of the quantitative data available were undertaken with the thought that some overall performance standard might be defined in terms of chemical permeability. There are, unfortunately, insufficient data relating FML physical property measurements to chemical permeability; thus, it was necessary to additionally consider what might be acceptable changes in some physical properties. Table 2 provides a listing of the chemical resistance criteria finally selected. (Note that HDPE has been assigned a separate criteria).

The criteria developed for assessing the chemical resistance of liner materials are based on the technical judgment of the investigators and are in general concurrence with the criteria that have been suggested by other investigators. A discussion of references may be found in the Study I report (Schwope et al., 1984). The criteria, however, have neither been proposed to, nor adopted as standards by, the EPA or any independent organization. They are believed to represent a good starting point for developing chemical compatibility criteria for liner materials. The values in Table 2 are not a guarantee nor are they intended to be used as a rule by which liners are either accepted or rejected for any given application. They are a set of values intended to provide a perspective from which information can be analyzed. Readers may also use the raw data provided in an appendix to the final report (Schwope et al., 1984) to derive their own conclusions.

Based on the normalized ratings reflecting qualitative data and the criteria for the evaluation of quantitative data (Table 2), an overall rating was assigned to each chemical-liner (or waste-liner) pair for which data were available. The ratings use the "+" and "-" symbols; one (either + or -)is assigned to represent compatibility conclusions derived from quantitative test data, and a second (also either + or -) is assigned to represent compatibility conclusions derived from qualitative information. A vertical bar, "|", is used to separate the two ratings for each pair. When data gaps are considered, the total number of ratings possible is eight (Table 3).

In all, the project report (Schwope et al., 1984) provides ratings for approximately 550 chemicals (or wastes) and 23 FML materials. In addition special discussions are given on FML-water interactions, environmental stress cracking, and the effect of chemical concentration on permeation rates through FMLs.

TABLE 1. COMPARISON OF THE CHEMICAL RESISTANCE INFORMATION FROM THREE HDPE VENDORS

	Normalized Rating (at 20-25°C)*		
Chemical	Vendor X	Vendor Y	Vendor Z
Carbon tetrachloride	bc	bc	bc
Glycerol	a	no data	a
Ethyl ether	bc	bc	bc
Aniline	a	a	a
Ethanol	a	a	a
Sulfuric Acid, <30%	a	a	a
Nitric Acid, <30%	a	a	a

*Ratings range from "a" most compatible to "d" least compatible; see text for discussion.

TABLE 2. CRITERIA USED TO DEFINE CHEMICAL RESISTANCE IN THIS STUDY

	Resistant (+)*		Not Resistant (-)*	
Property (units)	HDPE	Other FMLs	HDPE	Other FMLs
Permeation rate (g/m²d)	≤8.6	≤8.6	>8.6	>8.6
Change in weight (%)	≤3	≤10	>3	>10
Change in dimensions (%)	≤1	≤10	>1	>10
Change in tensile strength(%)	≤20	≤20	>20	>20
Change in % elongation at break (%)	≤20	≤30	>20	>30
Change in 100% or 200% modulus (%)	--	≤30	--	>30
Change in hardness (points)	--	≤10	--	>10

*The "+" and "-" symbols are the symbols used in the final compatibility matrix.

TABLE 3. RESISTANCE RATINGS USED FOR FML COMPATIBILITY DATA

Rating	Test Data		Qualitative Information
+|+	Good resistance to chemical	and	Favorable ratings
+|	Good resistance to chemical	and	No qualitative ratings
|+	No test data	but	Favorable ratings
+|-	Good resistance to chemical	but	Unfavorable ratings
-|+	Poor resistance to chemical	but	Favorable ratings
-|	Poor resistance to chemical	but	No qualitative ratings
|-	No test data	but	Unfavorable ratings
-|-	Poor resistance to chemical	and	Unfavorable ratings

STUDY II - TEST METHODS FOR DETERMINING THE CHEMICAL WASTE COMPATIBILITY OF SYNTHETIC LINERS

INTRODUCTION

It is well known that no one synthetic liner is resistant to all chemicals, that most facilities receive a variety of wastes, and that the nature of these wastes is likely to change with time. In some cases, the exact composition of the waste for a proposed treatment or disposal facility may not be known. However, for chemicals of known composition, there is a need to know what tests can be conducted to: (1) select a chemically-resistant liner, and (2) demonstrate (to the public and permit-granting authorities) that long-term resistance and compatibility are likely.

A variety of waste-liner compatibility tests have been described in the project report for Study II (Tratnyek et al., 1984). Some have been proposed by regulatory agencies, while others have been put forth by national standards-setting organizations, individual companies in the liner technical community (resin manufacturers, sheet manufacturers), research organizations and foreign countries. Among this group of available test protocols there is not a universally accepted compatibility test method.

Although details of the available tests vary, all are laboratory tests in which selected physical properties of the FML are compared and evaluated after contact with the chemical or waste for a specified period of time and under specified conditions (e.g., temperature). All tests are tedious, time-consuming, and expensive. Useful data for product specification and application are derived from these tests, but none adequately addresses all issues and questions raised, including chemical permeation of the liner and liner life-time prediction.

For the benefit of regulators, planners, and designers, the current test methodology requires review and assessment so that appropriate tests can be selected, the value of data from any particular test assessed, and improvements made.

PURPOSE

The purpose of this Study was to compile, review, and evaluate available test methods for measuring and/or estimating the chemical compatibility of liquid wastes with polymeric flexible membrane liners (FMLs) to be used at hazardous waste treatment, storage and disposal facilities.

The intention was not write a test method for FML/waste-compatibility testing, but to assess the current state of affairs and direct the thinking of all those concerned towards routes that might lead to a satisfactory, comprehensive, and reliable chemical compatibility measurement technique.

APPROACH

In carrying out this task, the focus has been placed on identifying and describing not only tests designed to assess FMLs, but also tests from related technologies (e.g., for pipe, packaging, and film) that are used to evaluate the compatibility of chemicals with plastics and other organic polymers. All types of tests that were relevant to chemical resistance and the transfer of liquids through membranes were considered and reviewed.

Levels of information sources are categorized as follows:

I. International standards and tests;
II. National standards and tests;
III. Industrial standards and tests;
IV. Project tests; and
V. Academic/literature tests.

The organizations, along with the identification code used in the text, for which tests are reviewed are listed below:

International Organization for
Standardization (ISO)
Geneva, Switzerland
Deutsches Institut fur Normung (DIN)
Berlin, Germany
British Standards Institution (BSI)
London, England
U.S. Environmental Protection Agency
(EPA) Washington, DC
National Sanitation Foundation (NSF)
Ann Arbor, MI
American Society for Testing
Materials (ASTM)
Philadelphia, PA
U.S. Military Standards (MIL)

Information from these organizations and sources was obtained by a combination of personal contacts and a literature search using computerized bibliographic data bases.

As part of the need to elucidate and understand compatibility test methods and FML requirements, a meeting with experts in FML technology and applications was held at Arthur D. Little, Inc. (Cambridge, MA), in January 1984. The meeting was attended by three FML manufacturers' representatives, an independent FML researcher, EPA representatives (Office of Solid Waste and Office of Research and Development), and members of the Arthur D. Little, Inc. staff. This meeting provided an opportunity to discuss the current test methodology and recommendations for improvements.

RESULTS

Information was obtained on 27 compatibility tests (Table 4). The project report (Tratnyek *et al*., 1984) provides a written description and a flowchart of each test. The study was intended to document the parameters of the chemical challenge to the FML and the subsequent measurements and observations made. The range of possibilities involved in such compatibility tests is shown in Figure 1.

Test methods are more or less similar and the basic approach appears to be patterned after previously developed test methods used for rubber and plastic sheet. However, a caveat is often given, warning that the test may not correspond with the service condition, and that the data are only comparative in nature. Absolute values or criteria for compatibility are not provided by the tests. Thus, interpretation of test results remains a very subjective undertaking based upon experience and an understanding of FML properties.

The tests reviewed generally exhibited the following features:

TABLE 4.
LISTING OF COMPATABILITY TESTS REVIEWED

I. INTERNATIONAL STANDARDS AND TESTS

ISO 175 Plastics -- Effect of Liquid Chemicals Including Water
ISO 1817 Vulcanized Rubbers -- Resistance to Liquids
DIN 53 521 Rubber -- Resistance to Liquids, Vapors and Gases
DIN 53 532 Permeability of Elastomers to Liquid Fuel
BS 4618 Chemical Resistance of Plastics to Liquids
BS 5173 Hoses -- Chemical Resistance

II. NATIONAL STANDARDS AND TESTS

EPA Method 9090
NSF Standard 54
ASTM D543 Resistance of Plastics to Chemcial Agents
ASTM D814 Rubber -- Vapor Transmission
ASTM G20 Chemical Resistance of Pipeline Coatings
ASTM D471 Rubber -- Effect of Liquids
MIL-T-6396D Aircraft Tanks

III. INDUSTRIAL STANDARDS AND TESTS

Schlegel Lining Technology, Inc.
J.P. Stevens and Company, Inc.
Gundle Lining Systems, Inc.

IV. PROJECT TESTS

Simulation Test (Haxo)
Pouch Test (Haxo)
Tub Test (Haxo)
Immersion Test (Haxo)
NSF FML Project Short- and Long-term Screening Tests
Harwell, England -- Accelerated Tests for HDPE

V. SELECTED ACADEMIC/LITERATURE TESTS

Sequential chemical absorption (Carpenter and Fisher)
Fluid Resistance of rubbers and Elastomers (Minter and Meier)
Stress Cracking by Creep Rupture Test (*Modern Plastics Encyclopedia*)
Chemical Stress Relaxation (Okuda)
Proposed -- Environmental Stress Cracking of Ethylene Plastics (Crissman)

- Immersion or one-sided exposure of FML test specimen to waste;
- Exposure of test specimen without stress, except for the polyethylene-type plastics (which are a special case);
- Exposure at ambient temperature and some elevated temperature (the latter to provide an accelerated test);
- Exposure of varying time (short to long), or to point of no change in mechanical properties;
- Test in laboratory;
- Exposure to reagents or waste;
- Measurement of mechanical properties and evaluation of appearances as indicators of compatibility.

At the present time, two specific laboratory test methods for evaluating FML performance in a chemical environment have evolved: NSF Standard 54 (NSF, 1983) and EPA's Method 9090 (EPA, 1984). The latter was designed specifically as a compatibility test for membrane liners in the presence of hazardous wastes. It has been proposed by EPA as the standard test to be used for all FML-waste compatibility data submitted in support of RCRA permit applications for lined facilities. The NSF method is a voluntary approach to industry standardization. Both tests rely on measurement of similar physical property changes (e.g., weight, dimensions, tensile strength, tear, elongation), and both employ similar test temperatures (23°C and 50°C), but neither specifies criteria for failure.

No one test satisfies all needs for FML-waste compatibility testing at RCRA sites. Indeed, there appears to be a need for at least three basic test protocols that could be used at different stages in a facility's life:

	Test 1	Purpose
I.	Short-term tests (up to 30 days' exposure in laboratory)	Preliminary Screening of liner materials
II.	Intermediate-term tests (up to 4 months exposure in laboratory)	Confirmatory tests for selected liner; data to be submitted with permit application

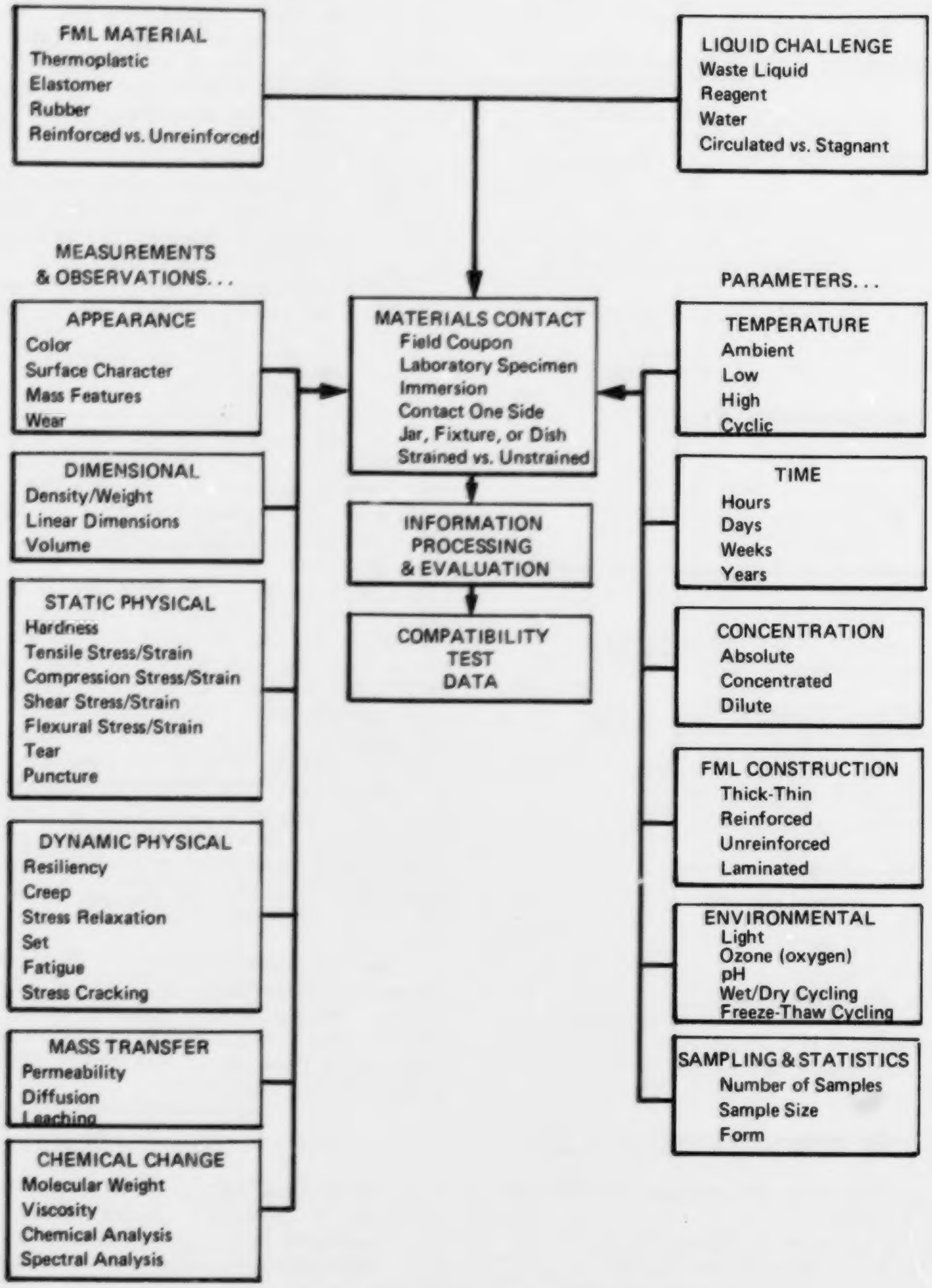

FML MATERIAL
Thermoplastic
Elastomer
Rubber
Reinforced vs. Unreinforced
LIQUID CHALLENGE
Waste Liquid
Reagent
Water
Circulated vs. Stagnant
MEASUREMENTS & OBSERVATIONS...
PARAMETERS...
APPEARANCE
Color
Surface Character
Mass Features
Wear
DIMENSIONAL
Density/Weight
Linear Dimensions
Volume
STATIC PHYSICAL
Hardness
Tensile Stress/Strain
Compression Stress/Strain
Shear Stress/Strain
Flexural Stress/Strain
Tear
Puncture
DYNAMIC PHYSICAL
Resiliency
Creep
Stress Relaxation
Set
Fatigue
Stress Cracking
MASS TRANSFER
Permeability
Diffusion
Leaching
CHEMICAL CHANGE
Molecular Weight
Viscosity
Chemical Analysis
Spectral Analysis
MATERIALS CONTACT
Field Coupon
Laboratory Specimen
Immersion
Contact One Side
Jar, Fixture, or Dish
Strained vs. Unstrained
INFORMATION PROCESSING & EVALUATION
COMPATIBILITY TEST DATA
TEMPERATURE
Ambient
Low
High
Cyclic
TIME
Hours
Days
Weeks
Years
CONCENTRATION
Absolute
Concentrated
Dilute
FML CONSTRUCTION
Thick-Thin
Reinforced
Unreinforced
Laminated
ENVIRONMENTAL
Light
Ozone (oxygen)
pH
Wet/Dry Cycling
Freeze-Thaw Cycling
SAMPLING & STATISTICS
Number of Samples
Sample Size
Form

Figure 1. The Compatibility Test Scheme

III.	Long-term tests (for life of facility) at actual site; use removable coupons	Monitoring actual durability of liner after exposure to environment and real wastes.

In addition to the development of such protocols as described above, research is required in other areas. There is as yet no reliable way to predict long-term performance of a FML from short-term and/or intermediate term tests (up to 4 months). There are no generally accepted rules for evaluating the test data that are generated. And tests, in general, must focus more on intended product performance and less on product specifications normally associated with the purchase of plastic sheeting. The whole scope of compatibility testing, and its relationship to performance criteria and their development, and to FML service lifetime predictions are subjects that require rethinking. New tests such as stress-relaxation and dynamic mechanical analysis may help in some cases.

III. STUDY III - CHEMICAL RESISTANCE RATING ESTIMATION: SOLUBILITY PARAMETER APPROACHES

INTRODUCTION

The availability of FML-waste compatibility tables summarizing data from tests of materials immersed in the chemicals/mixtures of concern or from permeation testing would be useful in selecting suitable liner materials for a given waste. Unfortunately, the amount of data available is extremely small relative to the large variety of possible exposure conditions. Furthermore, testing is not always practical within the time and budget constraints of many projects. There is a need, therefore, to develop analytical techniques that will predict the chemical compatibility (i.e., resistance) of polymeric materials upon their exposure to chemicals and chemical mixtures.

APPROACH

In the initial stages of Study III (Schwope _et al_., 1985) a literature search was undertaken to identify areas of proven success, and areas of needed research, in predicting chemical compatibility with polymers. As a result of the literature search, a decision was made to focus the remainder of the program on several predictive techniques based on solubility parameter theory, and statistical analysis and correlation of chemical properties with chemical resistance.

All of the studies and analyses carried out were based upon literature data on solubility parameters and upon manufacturers' ratings of chemical resistance. No laboratory studies were undertaken.

RESULTS

This program was completed in early 1985, and the draft final report (Schwope _et al_., 1985) is currently being reviewed by the EPA. A summary of some of the more important findings is presented below.

Solubility Parameter Theory

Solubility parameter theory has been used as a very effective tool for predicting chemical compatibility in solute-solvent systems. According to Hildebrand and Scott (1950), the overall solubility parameter of a chemical, δ, is its cohesive energy density:

$$\delta = [(\Delta H^{vap} - RT)/V]^{\frac{1}{2}} \qquad (1)$$

where ΔH^{vap} is the molar enthalpy of vaporization, R is the universal gas constant, T is temperature, and V is liquid molar volume. δ has units of $(\text{pressure})^{\frac{1}{2}}$. According to the theory, substances with similar solubility parameters ($\delta_{(A)} - \delta_{(B)} < 3$) will enter readily into solution along the "likes dissolve likes" axiom, while materials with strongly differing solubility parameters will be mutually insoluble.

As an expansion on this idea, Hansen (1967) developed the three-component solubility parameter. These three components take into account the major interactive forces of a chemical: dispersive forces (D), polar forces (P), and hydrogen bonding forces (H). According to Hansen (1967):

$$\delta_T^2 = \delta_D^2 + \delta_P^2 + \delta_H^2 \qquad (2)$$

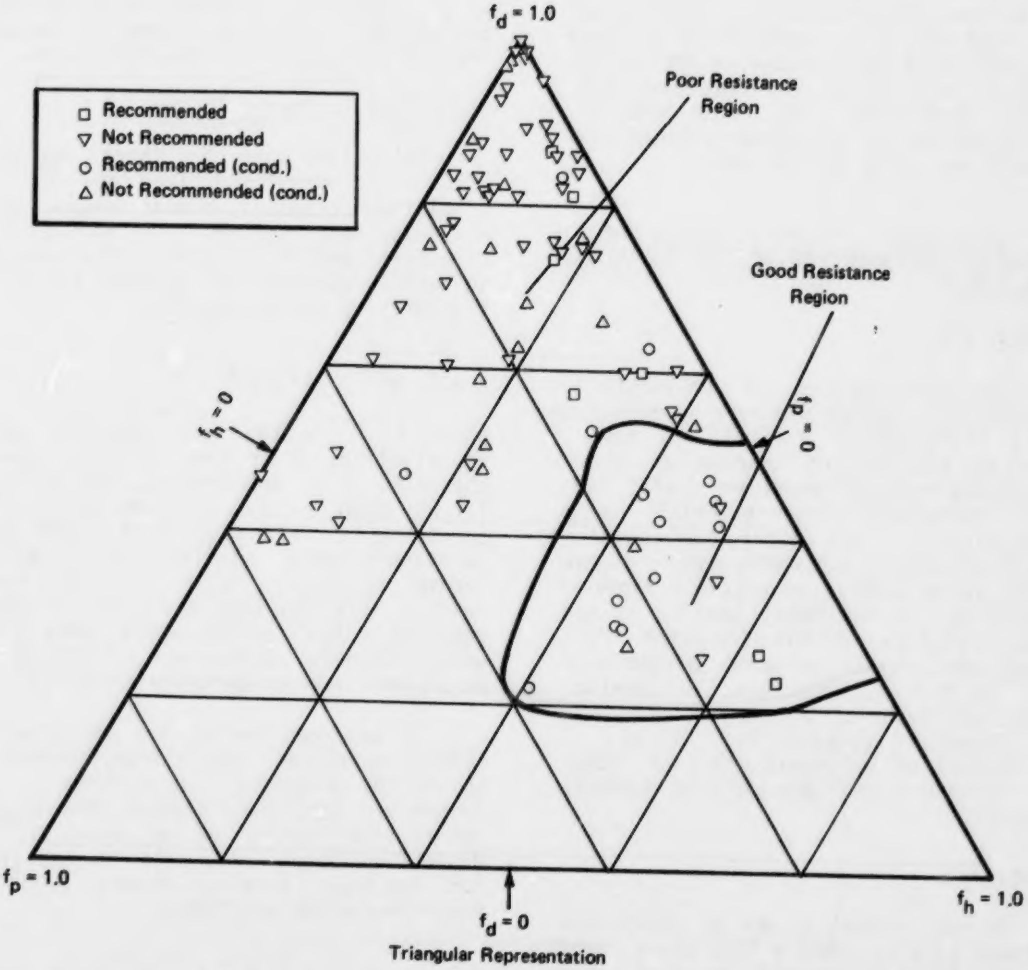
For many applications, the three-component approach has been shown to be a better predictor of solute-solvent behavior than the single-parameter approach, and it has been widely investigated and adopted.

Solubility Parameter Maps

One well-established graphical approach for examining chemical compatibility is the Solubility Parameter Map. These "maps," which can be drawn in two or three dimensions, allow estimation of chemical compatibility visually, based only on the three-component Hansen parameter of a given chemical. They can be created for virtually any resistant material for which enough compatibility data exist in order to confidently establish regions of common chemical resistance.

Figure 2 provides an example of a three-dimensional solubility parameter map for plasticized polyvinyl chloride. The coordinates for each chemical were calculated by the normalization of each of the three solubility parameters components to a scale of 0-100, i.e.:

$$f_i = \frac{\delta_i^2}{\delta_D^2 + \delta_P^2 + \delta_H^2} \times 100 \quad (3)$$



Triangular Representation

Figure 2. Three-Dimensional Solubility Parameter Map for PVC

Chemicals with different degrees of compatibility, ranging from "excellent" to "not recommended", are represented by different symbols on the map. (Note that the location of the point for any chemical on such a map is independent of the polymeric material; only the compatibility rating may change.) In the case of Figure 2, two clear regions of different compatibility show up, one of poor resistance and one of good resistance; there is also a large area for which no information is available. To the extent that reasonably clear areas of good and poor resistance can be identified on such maps, our ability to predict resistance for new untested chemicals is greatly enhanced.

In a roughly similar manner, two-dimensional solubility parameter maps may be created by plots, for example, of δ vs. δ_H and δ_P vs δ_H. Again chemicals with different degrees of compatibility are represented on the plot (cartesian coordinates this time, not triangular) with different symbols and the degree of clustering evaluated.

Statistical Correlation

A statistical approach was undertaken in an attempt to correlate solubility parameters and other fundamental chemical properties with material vendors' rating of chemical resistance. Three resistant materials were examined: CPE, Neoprene, and Butyl Rubber. Using the vendors' ratings, established from swelling data or similar tests, we placed each chemical into one or two categories; "Compatible" or "Not Compatible" with each of the liner materials. Using a computerized statistics package, the categories were then correlated with the three components of the Hildebrand solubility parameter, as well as such fundamental chemical properties as molecular weight, molar volume, dipole moment, boiling point, and molecular connectivity.

The statistics program employs a fundamental linear routine which established coefficients for each of the parameters involved. These coefficients are then used to establish the probability of a particular chemical falling into each of the two rating groups. Based on these probabilities, each chemical is assigned a chemical resistance rating, and this new, correlation-based rating is compared with the original manufacturer's rating, and the accuracy of the correlation assessed.

In one test of CPE resistance, for example, a data set of 123 chemicals (86 considered "Compatible" and 37 "Not Compatible") was evaluated using the three components of the solubility parameters (δ_D, δ_P and δ_H). The resulting correlation is described by equations 4, 5, and 6:

$$D_1 = -86.88 + 10.85\,\delta_D - 0.215\,\delta_P + 0.072\,\delta_H \quad (4)$$

$$D_2 = -109.69 + 12.88\,\delta_D - 0.291\,\delta_P - 0.089\,\delta_H \quad (5)$$

$$P_1 = \exp(-.5\,D_1^2)/[\sum_{i=1}^{2} \exp(-.5\,D_i^2)] \quad (6)$$

where D_1 and D_2 are discriminant values for chemicals in the "Compatible" and "Not Compatible" classes, respectively, and P_1 is the probability ($0 < P_1 < 1$) of a chemical being "Compatible." In a test of this correlation, chemicals with $P_1 > 0.5$ were considered "Compatible." In this manner, the correlation was able to correctly classify approximately 80% of the chemicals in the test set to the correct compatibility class. When molar volume and liquid density were added as correlation variables, no appreciable improvement in correlation accuracy was observed.

Exchange Energy Density Parameter

The exchange energy density parameters, A^{ij}, is a second non-graphical method for assessing chemical compatibility of resistant materials. A^{ij} is a measure of the change in energy density associated with the process of mixing component "i" with component "j," and it is calculated with three-component solubility parameters. The exchange energy density parameter simply combines the difference between the solubility parameter values of the resistant material (polymer) and the solubility parameter values of the solvent for each of the three Hansen components:

$$A^{ij} = [\delta_D{}^i - \delta_D{}^j]^2 + [\delta_P{}^i - \delta_P{}^j]^2 + [\delta_H{}^i - \delta_H{}^j]^2 \quad (7)$$

where in this case the superscripts "i" and "j" represent the polymer and the chemical, respectively. Values of A^{ij} close to zero are indicative of two materials that are likely to enter into solution ("Not Compatible" in the context of this study), while higher A^{ij} values are indicative of chemical differences that are likely to exhibit themselves by mutual insolubility ("Compatible" in the context of this write-up).

In using A^{ij} to estimate a polymeric material's performance for a particular chemical, the three-component solubility parameters for both the chemical and the polymer must be known. Three-component parameters for several hundred chemical solvents are available from the literature or can be estimated by group contribution techniques. However, solubility parameters for most polymeric materials are not readily available. Furthermore, little is published regarding the effects of compounding ingredients on the solubility parameter of commercial polymeric materials. Thus it often becomes necessary to estimate the solubility parameter of polymer materials from swelling test or other analytical methods. One relatively easy method of rough estimation involves the 3-dimensional solubility parameter maps discussed earlier. From the maps, one can estimate the point of "least compatibility" as the center of the "Not Recommended" region. This point will correspond to maximum solubility (i.e., least chemical compatibility), and thus will be indicative of the solubility parameter components for the polymeric material. The fractional parameter values (f_i) can be read off the map, and δ estimated by using the δ value of the chemical plotted closest to the maximum solubility point. From this information, δ_D, δ_P, and δ_H, for the polymer can be calculated.

This study has evaluated the potential of the exchange energy density parameter approach for CPE and HDPE. Figure 3 shows, for example, the results for CPE using data for 113 chemicals. In this figure, the percentage of chemicals rated as either "Recommended" or "Not Recommended" within eight different regions of A^{ij} values (0-2, 2-4, 4-6, etc.) is shown. The Figure shows clearly that when a chemical has an A^{ij} value above 8 (calculated for CPE), it is almost certain to be compatible with the liner material. Although the results for HDPE are not as sharply defined as those for CPE, they do show discernible trends.

ACKNOWLEDGMENTS

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The principal investigator for Study I and Study III is Arthur Schwope. The principal investigator for Study II is Joseph Tratnyek.

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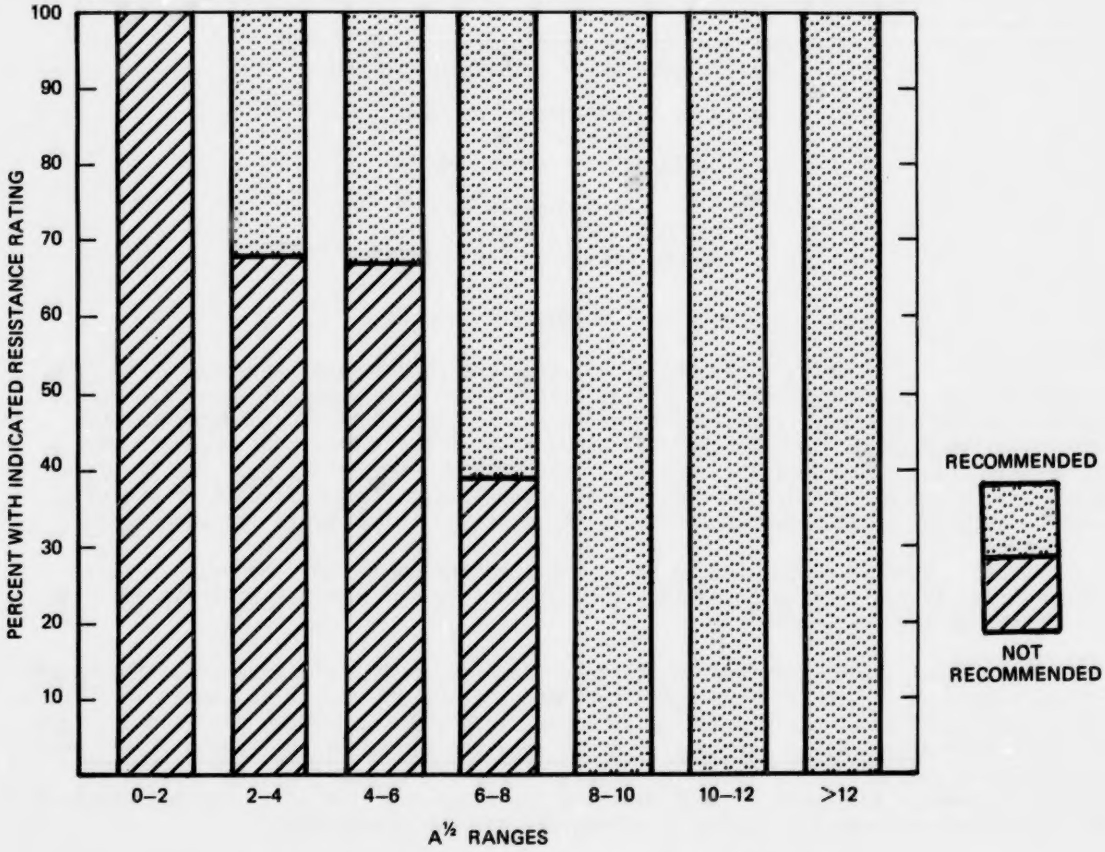

Figure 3. Change in Chemical Compatibility Ratings for 113 Chemicals with CPE as a Function of the Exchange Energy Density Parameter, A

RECOVERY AND TESTING OF A SYNTHETIC LINER FROM A WASTE LAGOON AFTER LONG-TERM EXPOSURE

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ABSTRACT

Samples of a 100-mil, high-density polyethylene (HDPE) liner were recovered from a waste lagoon in the northeastern United States after 4.75 years of service. Samples exposed to the waste were removed from different depths and tested to determine the effects of the exposure on the physical properties of the liner. The recovery was performed during closure of the lagoon in a Superfund Remedial Action. The impounded waste liquid was predominantly aqueous and contained significant amounts of organics, particularly chlorinated hydrocarbons, which increased in concentration with depth.

Overall, the liner appeared to be in satisfactory condition. No evidence indicated that it had cracked or failed, but it did show considerable waviness and distortion on the berm and slopes.

Liner samples were analyzed and tested. Samples of the HDPE liner from the bottom of the lagoon showed an absorbed waste content of about 2%; they also showed a 10% loss in tensile strength at yield and similar losses in S-100 and S-200 moduli and 30% loss in modulus of elasticity. The samples taken from the slopes of the lagoon showed essentially no changes in physical properties. Construction equipment was used in an attempt to remove the waste without damaging the liner, but the liner on the bottom of the lagoon was nonetheless destroyed during the cleanup operations.

INTRODUCTION

The recovery of samples from a membrane liner at a waste lagoon in the northeastern United States provided an unusual opportunity to investigate a high-density polyethylene (HDPE) liner after 4.75 years of service in a rather severe environment. The lagoon had been used to impound wastes containing a variety of organic chemicals. Investigations of the performance of liners that have been in service are important to develop a data base on field experience, to verify laboratory studies on liner/waste compatibility, and to estab- the durability of liners under different service environments.

Samples of the 100-mil HDPE liner were obtained and tested when the lagoon, a chemical waste storage and bulking facility, was decommissioned and removed as part of a Superfund Remedial Action.

This report describes the recovery of HDPE liner samples from the lagoon and presents results of the analyses and testing of the samples. Also included is a brief description of the history of the lagoon and information on the wastes that that were in contact with the liner.

SITE HISTORY

Construction of the lagoon by the owners began with site preparation in 1978. The 100-mil HDPE liner was made in Germany and installed by the manufacturer. Installation began in the fall 1978, but was delayed until the following spring because of bad weather. The lagoon was completed in May 1979; it measured 117 by 122 ft and had a capacity of 700,000 gal. Most of the lagoon was above the natural grade. The depth ranged from 7 to 10 ft, sloping down toward the sump in the northwest corner. The sump consisted of a 2-ft-diameter vertical HDPE pipe in the lagoon bottom connected to an 8-in.-diameter pipe that passed under the dike wall to the outside of the lagoon and connected to a 3-ft-diameter vertical HDPE pipe.

According to the decommissioning justification document, the U.S. Environmental Protection Agency (EPA) became involved in January 1981. The owner of the facility had declared bankruptcy and abandoned the site. An emergency was declared, and clean up work began immediately. The lagoon was sampled and drawn down, as it was close to overflowing. Drums of hazardous wastes on the site were stabilized. In October 1982, a remedial action master plan was completed for the site, and a fast track feasibility study was prepared for the lagoon.

Information from the decommissioning justification document indicates that some wastes were dumped into the lagoon before the HDPE liner was installed. The document also indicates that the lagoon overflowed during its use.

ANALYSIS OF THE WASTE

The decommissioning justification document indicated that in July 1983, the lagoon waste had been sampled from a small boat. During that sampling, sediment samples were collected from each of the four corner areas (see Figure 1). Aqueous waste samples were collected at the northwest corner, the center, and the southeast corner (see Figure 1). Different depths were sampled at each point.

The sediment that was directly on the lagoon liner underwent limited testing. Data on its physical and chemical properties (Table 1) show the range in sediment composition at the four sampling points. The lagoon sump is the deepest point of the lagoon, and the southwest corner is the shallowest. The low flash point and high energy value for the sump sediment indicate the presence of organic solvents, which tend to be aggressive to most lining materials. These samples were extracted according to the Extraction Procedure (EP), and the extract was analyzed for primary drinking water parameters.

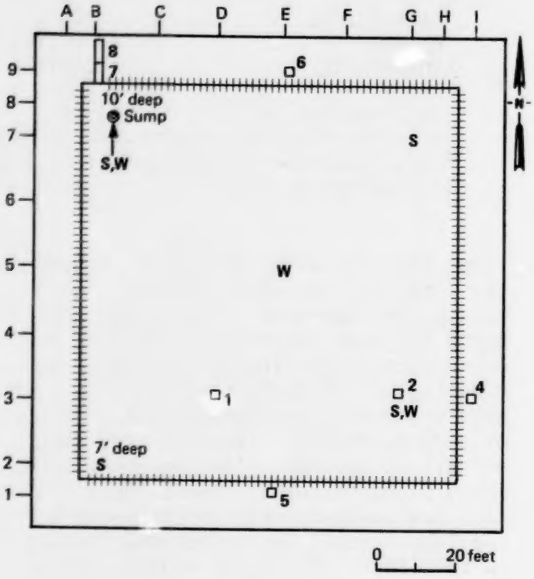

Figure 1. Lagoon lay-out showing grid pattern used in lagoon sampling, points of sample collection, and location of sump ◎. Liner samples are indicated by ☐ with the liner sample number, sediment samples are indicated by S, and aqueous waste samples are indicated by W. The toe of the slope is indicated by a hashed line (+++++).

The aqueous waste samples were analyzed for more parameters. Tests were run for inorganic and organic priority pollutants, priority pollutant pesticides, PCB's, and other inorganics. Samples were taken from three or four depths at each of the three collection points. Concentrations of most constituents increased with depth, showing that the lagoon waste had become stratified. Data are available for all samples. Those from the greatest depth at each collection point are of greatest interest in terms of the liner,

TABLE 1. PHYSICAL AND CHEMICAL PROPERTIES OF SEDIMENT SAMPLES COLLECTED FROM THE LAGOON SITE ON JULY 25-26, 1983[a]

Parameter	Point of sampling			
	Lagoon sump	B-2	G-7	G-3
pH	5.4	8.6	10.3	10.3
Flash Point, °C	26	>60	>60	>60
Corrosivity	Noncorrosive	Noncorrosive	Noncorrosive	Noncorrosive
Ignitability	Ignitable	Not ignitable	Not ignitable	Not ignitable
Energy value[b], BTU/lb	7880	...	...	...
Total residue, %	19.84	56.83	7.89	13.19

[a] Date taken for the decommissioning justification document.
[b] Dry-weight basis.

since they represent the waste closest to the liner on the lagoon bottom. These samples are the most concentrated and hence the most aggressive. Because inorganic chemicals are not generally aggressive to polymeric membrane liners, the high concentrations (greater than 50 ppm) of copper, nickel, zinc, and iron should not affect the HDPE. On the other hand, phenols and petroleum hydrocarbons (71 and 2100 ppm, respectively, in the sample from the lagoon sump) could have some impact on the liner. The total organic carbon (TOC), which was greater than 6000 ppm in all three of the deepest samples, could also affect the liner. The analyses of organic priority pollutants indicated the presence of several aggressive chemicals at concentrations greater than 50 ppm in the sample from the sump (Table 2). Chlorinated solvents are known to cause swelling and deterioration of physical properties in HDPE liners. The analyses of aqueous waste samples from the greatest depth at the three sampling locations resulted in the detection of only minute amounts of the PCB Aroclor 1254 in two of the samples.

TABLE 2. COMPONENTS OF THE AQUEOUS WASTE AND CHARACTERISTICS[a] POTENTIALLY AGGRESSIVE TO THE LINER

Waste component or characteristic	Sampling point and depth[b]		
	G-3 3.0 ft	E-5 4.0 ft	Lagoon sump 6.5 ft
Organic priority pollutant, mg/L:			
Chloroform	2.65	3.2	56.6
Ethylbenzene	3.30	9.6	1080
Methylene chloride	102	140	325
Tetrachloroethane	1.7	3.8	182
Trichloroethane	28.6	90.2	9100
Toluene	10.4	24.5	522
1,1,1-Trichloroethane	0.31	...	50.0
1,2-Dichlorobenzene	0.96	1.89	151
Other:			
Phenols, mg/L	20	32	71
Total organic carbon, mg/L	6950	8230	17200
Petroleum hydrocarbons, mg/L	26.0	27.8	2100
pH	9.7	9.6	9.8
Flash point, °C	>60	>60	>60

[a] Data taken from the lagoon decommissioning document.
[b] Samples were taken from the greatest depths at three locations in the lagoon (see Figure 1).

LAGOON DISMANTLING

Remedial action on the lagoon began in mid-December 1983. The liquid contents of the lagoon were pumped into tanker trucks for treatment and transport to an approved disposal facility. Approximately one month was needed to remove the 100 truckloads of waste.

To solidify the remaining sludge for removal, ground corn husk was dumped at one corner of the lagoon and blended into the sludge with a wheeled bobcat (Figure 2). The wheeled vehicle and a toothless bucket were used initially to minimize damage to the lagoon liner. However, when it became obvious that the liner was being torn regardless of these precautions, a tracked vehicle with better traction was used. The combination of driving over the liner with the vehicles and snagging it with the bucket had severely damaged the liner on the lagoon bottom.

SAMPLING OF THE LINER FOR ANALYSIS AND TESTING

The liner was inspected and samples were collected for analysis and testing on February 2, 1984, when the solidifying operation was nearly complete. The day was sunny and cool; about 1 ft of snow covered the ground, the berm, and some of the lagoon slopes, especially on the south and west sides. Pieces of liner from the lagoon bottom were mixed in with the sludge and corn husks (Figure 3). The liner on the upper area of the lagoon slopes was distorted and buckled but in good condition. No apparent cracking or splitting was observed in that area, but in the lower area of the lagoon slopes, the liner was scratched and had many holes and tears (Figure 4). In one place, a rock had penetrated the liner. Samples of liner tears and creases were collected from these areas (Figure 5).

Two liner samples were removed from the sludge and corn husk mixture on the floor of the lagoon. An 18-in.-wide strip of liner was cut from the northwest corner of the lagoon; the strip ran from the bottom of the lagoon to near the top. The samples were wiped with hexane, wrapped in aluminum foil, and double-bagged in polyethylene bags. A sample of HDPE that had not been exposed to waste was cut from a roll of sheeting left over from the construction and stored just northeast of the lagoon (Figure 3). The outer, weathered layer was peeled back, and the sample was cut from inside the roll. The samples collected are described in Table 3, and

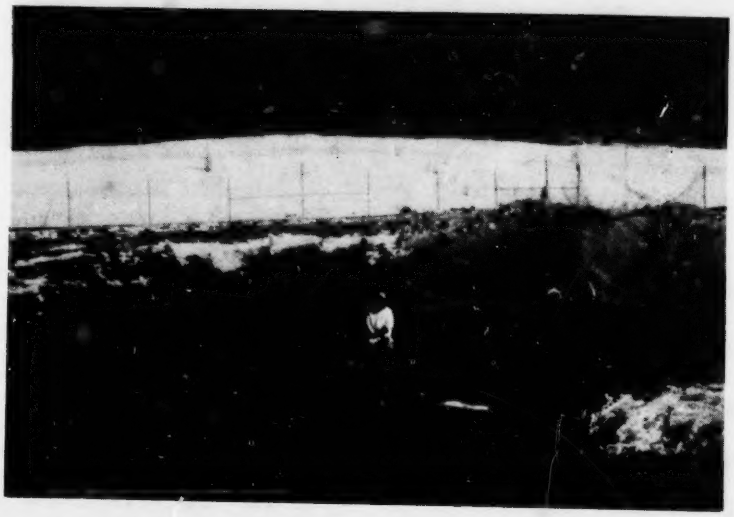

Figure 2. Blending of ground corn husk into the sludge with a bobcat. The husk had been dumped into the southeast corner of the lagoon.

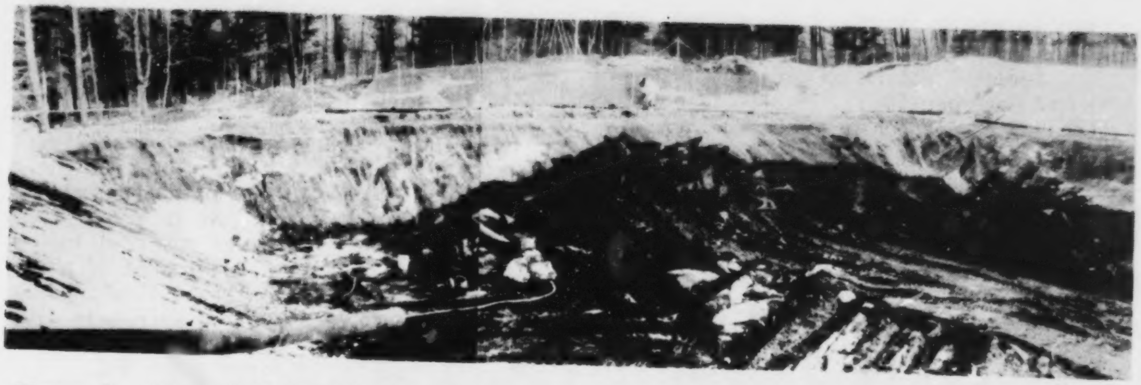

Figure 3. View of the north side of the lagoon. The sump is in the left (northwest) corner and is still in the process of being pumped out in this photo. Pieces of the liner from the lagoon bottom can be seen in the sludge/corn husk mixture. An unused roll of HDPE sheeting was stored beyond the right (northeast) corner between the two chainlink fences.

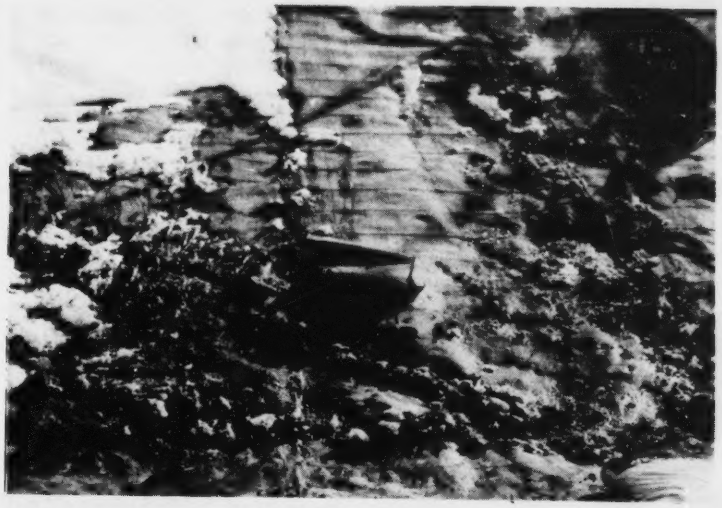

Figure 4. Extreme example of the holes and tears seen along the lower area of the lagoon slopes.

their locations in the the lagoon are shown in Figures 1 and 6.

The site was visited the following day for further observations. Much of the sludge and ground corn husk mixture was frozen. The liquid level at the sump had been drawn down enough to allow examination of the liner. Only a small portion of the seam or connection between the 2-ft-diameter polyethylene pipe and the liner was visible and intact.

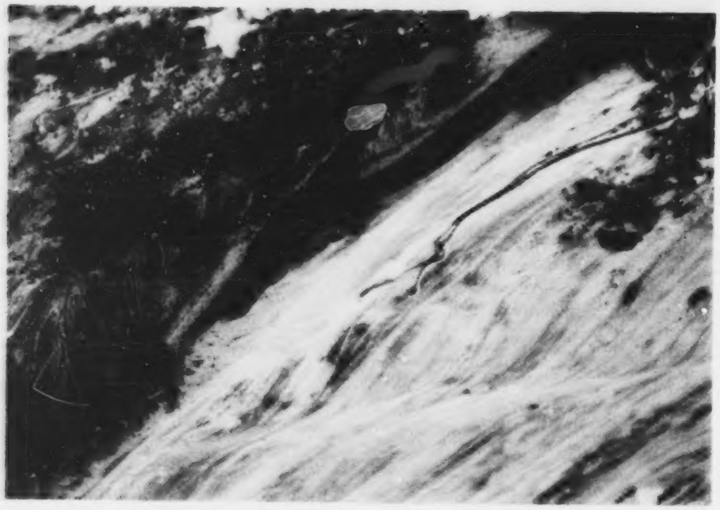

Figure 5. A pleated sample (Sample 6) in situ before it was cut from the lower slope of the liner in the center of the north side.

TABLE 3. SAMPLES COLLECTED FROM THE 100-MIL HDPE LAGOON LINER

Sample number	Location in lagoon	Size, in.	Feature
1[a]	Lagoon bottom, south-center	14x16	...
2	Lagoon bottom, southeast corner	10x19	...
3[a]	Retained sample from roll stored on site, northeast of lagoon	10x15	...
4	Lower slope, southeast corner	10x12	Sample with crack/tear
5	Lower slope, south side	14x11	Sample with tear next to extrusion line
6[a]	Lower slope, north side	6x26	Pleated sample with two sharp creases
7[a]	Bottom to midway of slope, northwest corner	18x69	Strip cut from bottom to top, bottom half
8[a]	Midway to top of slope, northwest corner	18x114	Strip cut from bottom to top, top half

[a] Analyzed and physically tested.

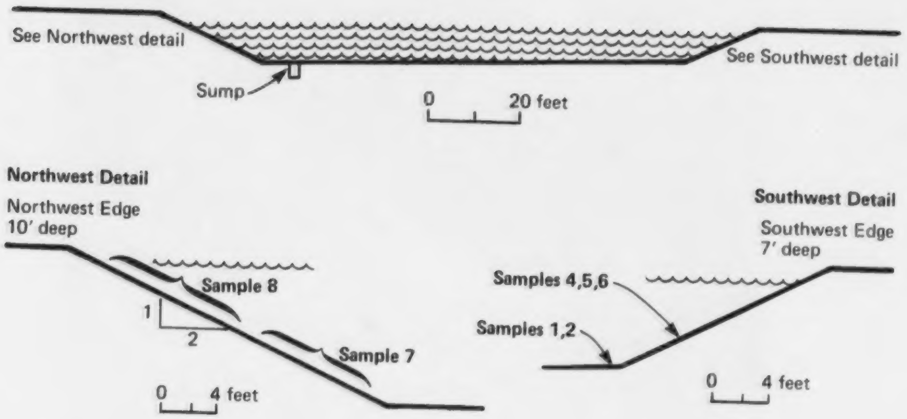
See Northwest detail
Sump
0
20 feet
See Southwest detail
Northwest Detail
Northwest Edge
10′ deep
Sample 8
1
2
Sample 7
0
4 feet
Southwest Detail
Southwest Edge
7′ deep
Samples 4,5,6
Samples 1,2
0
4 feet

Figure 6. Cross section of the lagoon from the northwest to southwest corners. The approximate depths at which liner samples were collected are shown in the details.

SAMPLE TESTING

Analytical and Physical Testing

The liner samples were photographed, measured, and diagrammed upon receipt at the laboratory. Descriptions of these samples and their respective locations in the lagoon are presented in Table 3. The following samples were selected for full testing:

- Sample 1, exposed liner from bottom of lagoon.
- Sample 3, retained, unexposed membrane left in roll on site.
- Sample 7, cut from the lower end of the strip, i.e. at the slope bottom.
- Sample 8-mid, cut from the middle section of the strip, midway up slope.
- Sample 8-top, cut from the top of the strip at top of slope.

These samples were tested as follows:

Property	Test method[a]
Tensile strength	ASTM D638
Modulus of elasticity	ASTM D882
Tear strength	ASTM D1004
Puncture resistance	FTMS 101C M2065
Hardness	ASTM D2240
Volatiles content	MTM 1
Extractables content	MTM 2

Results of the analytical and physical testing are presented in Tables 4 and 5. To assess the effect of depth on the volatiles content of the liner, additional testing was performed on specimens cut at 2-ft intervals along the strip, consisting of Samples 7 and 8 (Table 6).

Careful inspection of the sample edges revealed that the sharp crease of the pleated fold (Table 3, Sample 6) was probably made by the blade of the bobcat when it hit a wavy distortion of the liner. Tensile tests of specimens cut across the pleat yielded low tensile strength values and breaks at yield in the damaged area.

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[a] ASTM = American Society for Testing and Materials; FTMS = Federal Test Method Standard; MTM = Matrecon Test Method (Matrecon, 1983).

TABLE 4. PROPERTIES OF HDPE LAGOON LINER AFTER APPROXIMATELY 4.75 YEARS IN SERVICE

Properties	Direction of test	Unexposed sample of similar HDPE ...	Sampling location in lagoon and sample number				
			Retained sample[a], No. 3	Slope top, No. 8-top	Slope midway up, No. 8-mid	Slope bottom, No. 7	Lagoon bottom, No. 1
Analytical properties:							
Volatiles, total loss, %		0.06	0.15	0.22	0.56	1.00	2.26
Over desiccant at 50°C		...	0.12	0.15	0.30	0.64	1.90
In oven for 2 h at 105°C		...	0.03	0.07	0.26	0.36	0.36
Extractables, %		...	0.00	1.26	0.80	1.18	0.80
Physical properties:							
Thickness, mil		103.0	102.3	93.4	103.6	100.6	92.4
Tensile at yield, psi	Machine	2445	2705	2720	2725	2650	2445
	Transverse	2440	2700	2835	2700	2665	2440
Elongation at yield, %	Machine	20	17	18	17	15	15
	Transverse	15	15	15	15	18	17
Tensile at break, psi	Machine	4635	3530	4810	4510	3605	3710
	Transverse	4445	4065	4355	4885	2735	3885
Elongation at break, %	Machine	1025	785	965	925	760	810
	Transverse	1010	860	875	985	640	845
Tensile set, %	Machine	900	680	840	805	655	700
	Transverse	890	750	760	860	550	730
S-100, psi	Machine	1765	1920	1960	1925	1875	1795
	Transverse	1710	1945	1965	1925	1965	1790
S-200, psi	Machine	1765	1930	1955	1920	1870	1835
	Transverse	1720	1940	1963	1925	1955	1790
Modulus of elasticity (psi x 10^4)	Machine	7.86	8.69	8.98	8.72	8.37	5.97
	Transverse	7.87	8.20	8.68	8.51	7.54	6.05
Tear, ppi	Machine	839	900	920	910	855	830
	Transverse	850	885	910	890	875	830
Puncture resistance:							
Gage, mil		98.5	106	98.5	101	100	93.1
Stress, lb		131	148	139	135	134	118
Elongation, in.		0.73	0.66	0.72	0.62	0.61	0.59
Hardness, Durometer points:							
Instant reading		...	60D	58D	59D	59D	58D
5-second reading		55D	57D	55D	56D	56D	54D

[a]Sample from unused roll of liner left onsite.

Three test dumbbells were cut across the pleat to include the two creases. As the pleat ran approximately in the machine direction, this constituted a transverse direction test. All three specimens broke at the top crease, which was the sharper of the two. Two of these specimens broke at the yield point, which was at 20% elongation. The average of this yield/break value was 2280 psi for the two specimens and was much lower than the value for the retained sample, which yielded at 2700 psi and broke at 4065 psi in the transverse direction. The third specimen broke at 780% elongation, yielded at 2490 psi, and broke at 3510 psi--still at a significantly lower value than those of the retained samples or of the samples taken at the same depth on the north slope. Heating the pleat at 105°C did not relieve the creases; they appeared to be permanent.

Thermogravimetric Analysis, Differential Scanning Calorimetry, and Specific Gravity

In addition to the volatiles and extractables analyses of the HDPE samples removed from the lagoon, thermogravimetric

TABLE 5. DATA ON THE RETAINED HDPE LINER SAMPLE FROM UNUSED ROLL AND PERCENT RETENTION OF PHYSICAL PROPERTIES FOR EXPOSED SAMPLES

Physical property	Direction of test	Retained sample, No. 3	Sampling location and sample number: Slope top, No. 8-top	Slope midway up, No. 8-mid	Slope bottom, No. 7	Lagoon bottom, No. 1
		Initial values	Percent retention			
Tensile at yield	Machine	2705 psi	101	101	98	90
	Transverse	2700 psi	105	100	99	90
Elongation at yield	Machine	17%	106	100	88	88
	Transverse	15%	100	100	120	113
Tensile at break	Machine	3530 psi	136	128	102	105
	Transverse	4065 psi	107	120	67[a]	96
Elongation at break	Machine	785%	123	118	97	103
	Transverse	860%	102	114	74[a]	98
Tensile set	Machine	680%	123	118	96	103
	Transverse	750%	101	114	73[a]	97
S-100	Machine	1930 psi	101	100	97	93
	Transverse	1945 psi	101	99	101	92
S-200	Machine	1930 psi	101	99	97	95
	Transverse	1940 psi	101	99	101	92
Modulus of elasticity	Machine	86900 psi	103	100	96	67
	Transverse	82000 psi	106	104	92	74
Tear	Machine	900 psi	102	101	95	92
	Transverse	885 psi	103	100	99	94
Hardness, Durometer points:			Change in points			
Instant reading		60D	-2D	-1D	-1D	-2D
5-second reading		57D	-2D	-1D	-1D	-3D

[a] Sample had many scratches.

TABLE 6. VOLATILES CONTENT OF SPECIMENS OF THE HDPE LINER TAKEN AT INCREASING DEPTH

Volatiles	Retained sample, No. 3	Sampling location along slope from top to bottom: Slope top, 0 ft[a]	Slope, 4 ft[a]	Slope, 6 ft[a]	Slope, 8 ft[a]	Slope, 10 ft[b]	Slope, 12 ft[b]	Slope, 14 ft[b]	Slope, 16 ft[b]	Lagoon bottom, 8 ft[c]
Total volatiles, %	0.15	0.22	0.24	0.35	0.51	0.56	1.08	1.26	1.00	2.26
Over desiccant at 50°C, %	0.12	0.15	0.12	0.14	0.29	0.30	0.73	0.82	0.64	1.90
In oven for 2 h at 105°C, %	0.03	0.07	0.12	0.21	0.22	0.26	0.35	0.44	0.36	0.36

[a] Cut from Sample 8 section of 18-in. wide strip taken from top to bottom of slope.

[b] Cut from Sample 7 section of 18-in. wide strip taken from top to bottom of slope.

[c] Cut from Sample 1 taken from the lagoon bottom which was about 8-ft deep at that point.

analyses (TGA) and differential scanning calorimetry (DSC) were performed. Specific gravity of the samples was also determined. The results are presented in Table 7 and are compared with those from the data base in our laboratory on HDPE sheeting from U.S. and German production. The following observations can be made:

- The sample from the lagoon bottom had a lower amount of crystallinity than the retained sample from the roll and the creased sample from the side slope.
- The retained and the creased samples had lower melting points and slightly higher crystallinity than the similar German sample.
- Overall, the data on the retained liner sample and those in the database on German membranes appeared similar to each other and different from those on the U.S.-produced sample.

Gas Chromatographic Headspace (GCHS) Analysis

As shown in Table 2, the impounded waste liquid contained significant amounts of organics that could be absorbed by HDPE to some extent. Data on volatiles and extractables confirm that a small amount of absorption took place. Also, the HDPE exposed at the bottom of the lagoon softened--that is, it lost in modulus and in hardness. Since some of the organics are volatile, GCHS and GC pyroprobe were explored to assess the absorption by the HDPE, at least qualitatively. A large variety of organics appeared to have been absorbed. Some of the volatiles specifically listed in the analyses in Table 2 were identified by GCHS. Some nonvolatile plasticizers were also identified.

DISCUSSION

The amount that the liner swelled (as evidenced by the volatiles content)

TABLE 7. COMPARISONS OF THE TGA, DSC, AND SPECIFIC GRAVITY OF THREE LINERS

	HDPE lagoon liner sample				A similar German liner		U.S. liner (domestic material)
Analytical properties	Bottom[a], No. 1	Top, No.8	Retained, No. 3	Creased[b], No. 6	Smooth	Rough	
Thermogravimetric analysis:							
Scan number	411	...	410	...	36	37	38
Volatiles, %	0	...	0	...	0	0	0
Polymer, %	97.6	...	97.7	...	98.1	98.6	97.7
Plasticizer, %	0	...	0	...	0	0	0
Carbon black, %	1.7	...	1.8	...	1.9	1.4	2.3
Ash, %	0.7	...	0.5	...	0	0	0
Differential scanning calorimetery:							
Scan number	411	...	412	419	15	16	12
Tm[c], °C	131	...	130	129	123	124	119
ΔH_f[d], cal/g	29.8	...	35.3	36.0	33.6	33.7	24.8
Crystallinity[e], %	43.6	...	51.6	52.6	49.1	49.2	36.2
Specific gravity	0.936	0.947	0.943	...	0.943	0.939	0.957

[a] Sample had thoroughly dried out during shelf aging before being tested.

[b] Sample taken at the apex of the crease in the pleat.

[c] Melting temperature of crystalline phase.

[d] Heat of fusion of crystalline phase.

[e] Percent crystallinity based on ΔH_f value of 68.4 cal/g for 100% crystalline HDPE.

increased with depth into the lagoon. The bottom sample (No. 1) contained about 2.3% volatiles, and the sample from the top of the strip (No. 8) contained approximately 0.2% volatiles. The retained sample from the roll (No. 3) also had about 0.2% volatiles. All five of the tested areas (the four exposed to waste and the retained sample) showed good physical properties, but the sample from the bottom of the lagoon (No. 1) was the softest (lowest durometer and modulus) and showed the lowest values for several physical properties. The sample from the bottom end of the slope strip (No. 7) had low values in tensile break in the transverse direction. This sample was observed during testing to be quite scratched, which probably explains the low values. Data for other liner properties from this area are much higher. The liner sample from the top of the strip (No. 8) showed higher tensile properties than the retained sample (No. 3).

CONCLUSIONS

The exposed liner sample from the lagoon bottom showed a small amount of swelling (about 3%) and 10% loss in both tensile at yield and in modulus. These changes do not demonstrate incompatibility, but they do indicate that the waste at the bottom of the lagoon had the greatest impact on the liner.

Much buckling and distortion of the liner was observed, especially on the north slope of the lagoon. The day the samples were taken was cold but sunny, which should have minimized any thermal expansion. On a hot day, this buckling would have been greatly increased. Though not in itself a significant problem, the buckling places added stress on seams, allows movement of the underlying earthwork by creating cavities, and increases the chance of mechanical damage by introducing folds.

The liner showed no cracking such as that encountered with environmental stress-cracking; it lost in tensile strength at yield and in ultimate tensile strength, but it did not crack at folds or bends.

No evidence was apparent to indicate that the HDPE liner lost its integrity. All visible seams looked secure. The only evidence of degradation was mechanical damage in areas where the liner had probably been worked on with the bobcat. The presence of rocks directly underneath the liner aggravated this damage. The liner in this impoundment was not able to withstand the clean-up operations.

DSC data indicate that the amount of polyethylene crystallinity in the HDPE liner changed during exposure, a factor that should be checked in future testing.

Decommissioning of liner waste impoundments under Superfund Remedial Actions affords an excellent opportunity to collect information on the performance and durability of lining materials. Efforts should be made to incorporate liner recovery and testing when such sites become available.

REFERENCE

1. Matrecon, Inc., Lining of Waste Impoundments and Disposal Facilities, SW-870, Revised (March 1983), U.S. Environmental Protection Agency, Washington, D.C. (GPO 055-00000231-2).

STRENGTH AND DURABILITY OF FLEXIBLE MEMBRANE LINER SEAMS AFTER SHORT-TERM EXPOSURE TO SELECTED CHEMICAL SOLUTIONS

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ABSTRACT

In March 1983, the United States Environmental Protection Agency (USEPA) began research with the United States Bureau of Reclamation (USBR) to evaluate flexible membrane liner (FML) seams exposed to selected environmental conditions over short periods. Thirty-seven combinations of reinforced and nonreinforced FML materials joined by various seaming methods have been subjected to six chemical solutions, brine, tapwater, and accelerated outdoor aging by ultraviolet light. Effects of these types of environmental conditions were evaluated using shear and peel strength tests before and after exposure. The tests were performed under dynamic load at room temperature, and static dead load at 50°C.

To date, tests have been completed on unexposed seams and seams that have been exposed for 3 months at room temperature. Exposures for 6 and 12 months are currently underway. The results and evaluation of the mechanical tests completed on 3-month chemical exposures are presented in this paper.

INTRODUCTION

Laboratory tests and pilot-scale studies have been conducted by various Government and non-Government group to assess the effects of chemical waste products on the integrity of FML's; however, little has been done to assess the performance of the various factory and field seams used in joining the manufactured roll goods in the factory and the panels seamed in the field.

The objective of this study is to evaluate bonded seams of commonly used FML materials and the bonding methods employed. Destructive tests are being conducted in an attempt to detect physical changes in these seams when exposed to selected chemicals and simulated environmental conditions.

This paper describes the liner samples which were obtained for study, field seam fabrication, the immersion test setup, and UV (ultraviolet) light exposure testing. Some preliminary results are presented on seam strength tests after short-term (3-month) chemical immersion at room temperature.

APPROACH

Materials Used in the Study

The polymeric materials evaluated for this study were obtained from 12 industry representatives who manufacture and/or fabricate FML's which are commonly used in pollution and seepage control. Manufacturers and fabricators were contacted and asked to participate in the study in an attempt to obtain a broad selection of recommended membrane types (including reinforced and nonreinforced samples), scrim, and seaming techniques.

Polymer sheet materials submitted by industry for this project include the following:

High Density Polyethylene (HDPE)
30-mil nonreinforced

High Density Polyethylene (HDPE)
80-mil nonreinforced

Linear Low Density Polyethylene (LLDPE)
30-mil nonreinforced

Ethylene Propylene Diene Rubber (EPDM)
30-mil reinforced

Polyvinyl Chloride (PVC)
30-mil nonreinforced

Chlorinated Polyethylene (CPE)
30-mil nonreinforced

Chlorinated Polyethylene (CPER)
36-mil reinforced

Chlorosulfonated Polyethylene (CSPER)
36-mil reinforced

Ethylene Interpolymer Alloy (EIA)
38-mil reinforced

Some sources submitted more than one sample and/or seaming method of PVC, HDPE, CPE, and CSPE. One composite seam of PVC/CPE was also submitted. Various scrim types, thickness, and seaming methods were represented in the submittals.

Field Seam Fabrication

In addition to the polymer roll goods and factory seams submitted by industry, field seams were constructed at the Bureau of Reclamation's Engineering and Research Center under representative field conditions.

The field seams were fabricated by a third party commercial installer in accordance with manufacturer's instructions and utilizing manufacturer-supplied equipment and products where required. Some field seams which could not be constructed at the Bureau's facility were witnessed and collected at job sites by Bureau personnel.

Factory and field seam types submitted or field-fabricated for evaluation in the project were as follows:

Thermal-hot air
Thermal - dielectric
Thermal - hot wedge
Extrusion - fillet weld
Extrusion - lap weld
Bodied solvent adhesive
Gumtape/cement
Vulcanized

Thirty-seven combinations of sheet materials and seaming methods are being evaluated in the chemical and water immersions and the accelerated outdoor aging.

Chemical and Aqueous Solution Immersions

All nonreinforced and reinforced-FML seams were cut into large samples for environmental exposure. The nonreinforced seams were cut into 300-mm (12-in) wide samples in order to later yield twelve 25-mm (1-in) wide specimens for mechanical testing and the reinforced samples were cut into 450-mm (18-in) wide samples to provide four 50-mm (2-in) wide and eight 25-mm (1-in) wide specimens.

To allow for possible edge effects due to scrim exposure, 25-mm (1-in) of each side of the scrim-reinforced samples were cut off and discarded. All samples were attached with polypropylene ties to fiberglass rods for exposure testing.

The six chemical solutions, chosen to represent a wide range of chemical groups, are:

10 percent phenol (organic acid),

10 percent hydrochloric acid
(inorganic acid),

10 percent sodium hydroxide
(inorganic base),

10 percent methyl ethyl ketone (ketone),

5 percent furfural (aldehyde),

100 percent methylene chloride (halogenated hydrocarbon).

Methylene chloride is not soluble in water; therefore, pure solvent was used to avoid the problem of phase separation. Pure chemicals or aqueous chemical solutions were selected for testing rather than simulated or actual wastes from waste sites to simplify verification of testing procedures. The use of representative groups of chemicals also allows for reasonable interpretation of the data.

Polyproplyene and polyethylene tanks, 45-gallon (170 L) capacity, were filled separately with each of the above chemicals, the seam samples were attached to the fiberglass rods, and the samples were then suspended into the chemicals.

In parallel with the tank immersions, smaller coupons of the liner materials are immersed in small clear glass jars for periodic weighings and thickness measurements. The smaller coupons allow for easier inspection of the polymeric sheet materials for obvious excessive degradation, swelling, or change in color or surface texture. If any accelerated response is observed in the coupons, the seam samples are removed from the larger tanks before they are destroyed completely. The ratio of the volume of liquid to the surface area for each coupon is 6.2 mL/cm^2 (40 mL/in^2).

Three tanks of room temperature tapwater and six tanks of saturated sodium chloride brine solution (three tanks at room temperature and three at 50°C (122°F) were also set up for immersing seam samples. Double-sided exposure of all samples was used to accommodate the large number of samples in minimum space. The accelerated effect of double-sided exposure compared to one-sided exposure reduces the time it would take to see effects the liquids may have on the samples.

Accelerated Outdoor Exposure

In addition to the above exposure conditions, the Desert Sunshine Exposure Test Laboratories in Phoenix, Arizona, have performed accelerated outdoor sunlight exposure testing on representative seam samples. The exposures were accomplished on the EMMAQUA(R) accelerated weathering test machine located at the test facility. The EMMAQUA(R) machines are capable of tracking the sun and focusing the sun's rays on the 5-inch-wide seam specimens for optimum UV exposure. One year of concentrated sunlight exposure is approximately equal to 5 to 7 years of actual sunlight exposure. After recent completion of exposure, the samples were visually inspected, photographed, and returned to the Bureau for testing to determine any change in peel strength. This testing had not been completed at the time of preparation of this paper.

Physical/Mechanical Testing

At the end of 3-month, 6-month, and 12-month chemical, brine, and water exposures and after completion of the accelerated outdoor exposure, the following physical and mechanical tests are being performed to determine changes in physical properties:

1. Volume (thickness) and weight - Coupon samples were measured for volume and thickness before immersion. Exposed coupon samples have been measured every week for the first month, monthly for the next 3 months, and quarterly thereafter.

2. Dynamic tensile shear testing - 50-mm (2-in) wide reinforced or 25-mm (1-in) wide nonreinforced seam specimens are tested in tensile shear using dynamic loading at a low strain rate of 0.8 mm/s (2 in/min). The reinforced samples are wider for tensile testing in an effort to eliminate edge effects due to the scrim reinforcement. Results are in N/m (lb$_f$/in) of seam width.

3. Dynamic peel testing - 25-mm (1-in) wide specimens are subjected to 180° peel at a strain rate of 0.8 mm/s (2 in/min). Results are in N/m (lb$_f$/in) of seam width.

4. Static dead load peel testing at elevated temperature - 22-mm (1-in) wide specimens are subjected to a 180° peel dead load equivalent to less than 5 percent of the material's breaking strength per inch of seam width at a

temperature of 50°C (122°F). Results will be expressed by pass/fail, time to failure, and mode of failure.

A summary of seam and peel strength under dynamic load, and failure mode for both original dry samples and samples subjected to 3-month chemical immersion is presented in Tables 1 and 2.

Discussion of Preliminary Results

There is considerable variation between seam types in the 3-month test results, as seen in Tables 1 and 2. Early in the exposure period, most of the seam samples immersed in methylene chloride (MCl) were destroyed and removed from the exposure tanks. Thus, the data in Tables 1 and 2 for MCl include only seven entries.

No attempt has been made to quantitatively analyze the results after 3-month exposure. After all data for all exposure periods have been stored on a computer file, a thorough analysis will be completed. However, qualitatively, it can be seen in these preliminary data that many of the failure modes were unchanged in samples before and after 3-month exposure to all the chemicals. In addition to MCl, it is evident that phenol had a deleterious effect on most samples after 3 months - both in shear and peel.

Some samples, particularly the PVC, show increased shear strength values due to the migration of plasticizer and subsequent stiffening after immersion in some of the chemicals. Samples that do well in shear, however, may not do so well in peel as evidenced by even these 3-month test results. The peel test is showing deterioration of bond after chemical immersion; whereas, the shear test may not readily indicate potential deterioration of bond.

After all test results are compiled and analyzed for the 3, 6, and 12 months of exposures, it is anticipated that definite aging trends will surface for the combinations of materials and bonding techniques examined in this research program.

Table 1. - Seam shear strength after 3 months exposure to six chemical solutions

Material	Seam type[1]	Original data		Immersion Media											
				Phenol - 10%		HCl - 10%		NaOH - 10%		MEK - 10%		HCl - 100%		Furfural - 10%	
		Shear (lbf/in)	Fail mode[2]	Shear (lbf/in)	Fail mode[2]	Shear (lbf/in)	Fail mode[2]	Shear (lbf/in)	Fail mode[2]	Shear (lbf/in)	Fail mode[2]	Shear (lbf/in)	Fail mode[2]	Shear (lbf/in)	Fail mode[2]
36 mil CPER	A	78.4	7	7.4	5	36.5	7	22.5	7	31.4	7			26.8	7
36 mil CSPER	A	90.4	6	16.4	5	40.5	7	31.7	7	45.0	7			43.5	5
36 mil CSPER	F	90.0	7	18.8	5	64.7	7	44.8	7	38.1	7			73.0	7
36 mil CPER	F	94.3	7	12.3	5	58.9	7	27.3	5	36.7	7			63.0	7
36 mil CSPER	A	31.0	7	24.7	7	31.1	7	24.8	5	32.0	7			34.8	7
36 mil CSPER	F	31.2	7	25.2	5	32.5	7	24.6	5	32.8	7			34.7	7
36 mil CSPER	A	63.8	7	24.9	5	50.5	7	35.0	7	45.3	7			46.8	7
30 mil CSPER	F	65.0	7	25.6	4	53.5	7	41.2	7	46.5	7			38.8	5
30 mil CSPER	A	79.9	7	32.8	5	139.0	7	42.0	7	57.2	7			71.5	5
30 mil CSPER	F	65.0	7	54.7	5	89.5	7	54.3	7	51.5	7			64.5	6
30 mil LLDPE	C	65.5	9	61.2	6	61.6	6	65.6	6	58.0	6	66.0	6	58.0	6
30 mil LLDPE	C	62.5	9	66.0	6	66.6	6	67.8	2	61.0	6	67.0	6	59.0	6
30 mil EPDM	I	70.2	7	58.2	5	88.0	7	67.6	7	82.0	7	111.4	7	92.0	7
30 mil HDPE	D	73.7	9	91.0	6	88.6	6	78.4	6	77.5	6	85.0	6	83.7	6
30 mil EPDM	H	91.4	7	61.3	5	58.9	7	44.2	5	70.9	7	-	-	69.0	7
80 mil HDPE	D	203.0	9	243.0	6	219.0	6	256.6	6	229.0	6	268.5	6	205.5	2
30 mil PVC	G	57.2	2	65.8	2	48.6	2	78.4	2	52.4	2	-	-	51.8	2
80 mil HDPE	E	242.0	9	267	6	265.0	6	256.0	2	247.0	6	253.3	6	263.0	6
30 mil PVC	G	47.7	2	60.5	2	52.8	2	81.4	2	49.4	2	-	-	66.0	2
80 mil HDPE	C	224.0	9	247.0	6	220.0	6	231.0	6	223.5	6	225.0	6	224.0	6
30 mil CPE	G	42.0	2	3.4	3	35.6	2	44.1	2	30.4	2			16.7	2
30 mil CPE	G	33.8	2	9.5	2	31.6	2	39.0	2	26.1	2			14.1	2
30 mil CPE	A	33.1	2	11.3	6	30.4	2	33.2	2	29.0	2			15.2	2
30 mil CPE	G	33.6	2	4.3	3	38.4	2	39.3	2	31.8	2			18.3	2
36 mil CSPER	B	73.5	7	28.0	5	70.9	7	79.2	7	61.3	7			100.0	7
30 mil PVC	A	52.9	2	71.8	6	53.2	2	123.5	6	50.1	2			53.8	2
36 mil CSPER	F	91.8	7	50.4	5	119.7	7	80.3	5	138.5	7			121.5	7
30 mil PVC	G	53.2	2	64.5	6	58.8	2	78.8	6	59.4	2			56.7	2
36 mil CSPER	F	118.7	7	29.8	5	86.3	7	98.0	7	80.5	7			97.0	7
30 mil PVC/CPE	A	38.2	2-CPE	12.7	3	26.0	2-CPE	33.3	2-CPE	18.8	2-CPE			15.9	2-CPE
36 mil CSPER	F	138.4	7	64.0	5	105.8	7	99.9	7	145.5	7			148.0	7
36 mil CPER	A	123.4	7	61.0	5	150.0	1	143.5	1	140.5	7			124.5	5
36 mil CPER	F	124.5	7	48.8	5	111.5	7	60.0	3	115.9	7			99.0	6
36 mil CSPER	F	97.4	7	38.8	5	124.9	7	80.0	5	122.5	7			144.8	7
30 mil PVC	G	59.6	2	66.4	2	62.6	2	76.0	2	67.5	2			66.0	2
38 mil EIA	A	353.8	6	120.1	3	9	9	9	9	9	9			9	9
38 mil EIA	A	356.6	6	9	9	9	9	9	9	9	9			9	9

[1] A: Thermal - hot air; B: Thermal - dielectric; C: Thermal - hot wedge; D: Extrusion - fillet weld; E: Extrusion - lap weld; F: Bodied solvent adhesive; G: Solvent adhesive; H: Gumtape/cement; I: Vulcanized.

[2] 1: Parent material failure at clamp edge; 2: Parent material failure at seam edge; 3: Within the adhesive bonding system; 4: Planar failure between adhesive and liner surface or between two thermal-bonded surfaces; 5: Scrim delamination and pullout; 6: Failure in parent material (other than 1, 2, or 5); 7: Scrim break; 8: Initial peak with subsequent ply adhesion; 9: No failure.

Table 2. - Seam peel strength after 3 months exposure to six chemical solutions

Material	Seam type[1]	Immersion Media													
		Original data		Phenol - 10%		HCl - 10%		NaOH - 10%		MEK - 10%		HCl - 100%		Furfural - 10%	
		Peel (lbf/in)	Fail mode[2]	Peel (lbf/in)	Fail mode[2]	Peel (lbf/in)	Fail mode[2]	Peel (lbf/in)	Fail mode[2]	Peel (lbf/in)	Fail mode[2]	Peel (lbf/in)	Fail mode[2]	Peel (lbf/in)	Fail mode[2]
36 mil CPER	A	59.8	7	8.2	4	36.0	5	35.8	7	33.8	7			30.8	7
36 mil CSPER	A	48.3	7	24.0	4	41.9	7	39.1	7	48.9	5			48.5	7
36 mil CSPER	F	53.4	6	30.4	4	63.6	7	54.2	7	67.8	6			74.3	7
36 mil CPER	F	71.8	6	21.5	5	65.7	7	45.7	7	56.1	7			66.8	7
36 mil CSPER	A	29.6	2	24.0	5	29.9	5	27.0	5	24.9	4			31.1	5
36 mil CSPER	F	31.7/22.5	8	23.1	5	30.5	5	26.1	5	30.9	5			14.7	3
36 mil CSPER	A	61.1	7	12.8	4	51.7	4	36.4	4	47.7	7			20.6	4
30 mil CSPER	F	31.0	3	2.9	4	12.2	3	11.6	3	3.8	3			7.4	3
30 mil CSPER	A	33.6/15.1	8	35.4/12.8	8	36.4	4	31.6/11.8	8	27.0	4			29.9/15.5	8
30 mil CSPER	F	32.8/17.7	8	38.2/21.8	8	35.1	4	27.7/17.5	8	24.9	4			27.2/15.5	8
30 mil LLDPE	C	64.4	9	64.2	6	61.7	6	65.7	2	58.8	6	2.70	6	61.7	6
30 mil LLDPE	C	60.6	9	70.1	6	64.8	6	65.1	2	59.7	6	3.10	6	64.2	6
30 mil EPDM	I	36.2/21.2	8	43.1/21.5	8	35.8	4	40.9	4	34.7	4	14.70/0.55	8	38.4/19.4	8
30 mil HDPE	D	72.3	2	87.0	6	80.6	2	69.0	2	65.4	6	7.80	2	86.3	2
30 mil EPDM	H	8.6	3	5.9	4	5.4	4	6.4	3	5.5	3	-	-	6.9	3
80 mil HDPE	D	123.0	2	143.3	2	77.5	2	97.2	2	158.7	2	41.00	2	149.3	2
30 mil PVC	G	14.8	4	23.3	3	15.4	3	25.4	3	15.4	3	-	-	4.5	3
80 mil HDPE	E	175.0	2	161.3	4	191.4	2	144.3	2	148.2	2	21.50	2	213.0	2
30 mil PVC	G	15.7	3	26.9	3	12.3	3	34.7	3	10.1	3	-	-	13.3	3
80 mil HDPE	C	120.8	4	150.3	4	169.8	2	152.7	2	170.4	2	9.50	2	169.2	2
30 mil CPE	G	21.3	2	<0.3	3	20.1	3	22.7	3	19.1	6			3.8	3
30 mil CPE	G	17	3	0.45	3	13.2	3	15.1	3	14.4	3			4.9	3
30 mil CPE	A	29.6	2	8.4	4	21.8	2	25.0	4	20.0	2			9.8	3
30 mil CPE	G	12.3	3	<0.3	3	14.2	3	12.7	3	11.7	4			4.1	3
36 mil CSPER	B	30.9/11.4	8	24.4/9.24	8	31.9/13.4	8	30.5/15.4	8	30.8/14.5	8			30.9/12.8	8
30 mil PVC	A	38.1	2	43.1	2	39.5	2	23.9	2	35.4	2			37.1	2
36 mil CSPER	F	34.4/17.7	8	25.5/17.8	8	25.3	7	29.3/16.3	8	25.7/15.3	8			28.9/17.0	8
30 mil PVC	G	19.2	3	13.7	3	21.5	3	27.6	3	18.5	3			8.8	3
36 mil CSPER	F	30.0/14.8	8	33.0/20.2	8	32.3/19.3	8	25.5/18.4	8	28.6/16.3	8			30.6/18.6	8
30 mil PVC/CPE	A	29.8	2-CPE	4.1	3	19.4	3	20.8	2	15.6	2			11.2	4
36 mil CSPER	F	31.7/15.1	8	21.8/12.8	8	30.2/15.4	8	26.8/16.4	8	22.3/14.0	4			18.0	3
36 mil CPER	A	16.2	4	7.5	3	20.2	3	26.6	4	12.4	4			25.8/10.5	8
36 mil CPER	F	23.4	3	3.4	3	19.5	4	-	-	17.3	4			17.4/24.5	8
36 mil CSPER	F	29.5/16.2	8	29.6/14.9	8	37.3/21.8	8	25.1/16.4	8	30.8/18.4	8			20.8	3
30 mil PVC	G	24.8	3	19.0	3	30.5	4	31.9	3	29.5	4			19.7	3
38 mil EIA	A	43.8/20.5	8	2.3	3	41.2/11.5	8	25.7/23.7	3	31.3/12.5	8			5.7	3
38 mil EIA	A	39.3/16	8	19.3	4	31.3/17.1	8	31.7/17.9	8	34.2/17.9	8			31.4/18.9	8

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[1] A: Thermal - hot air; B: Thermal - dielectric; C: Thermal - hot wedge; D: Extrusion - fillet weld; E: Extrusion - lap weld; F: Bodied solvent adhesive; G: Solvent adhesive; H: Gumtape/cement; I: Vulcanized.

[2] 1: Parent material failure at clamp edge; 2: Parent material failure at seam edge; 3: Within the adhesive bonding system; 4: Planar failure between adhesive and liner surface or between two thermal-bonded surfaces; 5: Scrim delamination and pullout; 6: Failure in parent material (other than 1, 2, or 5); 7: Scrim break; 8: Initial peak with subsequent ply adhesion; 9: No failure.

CHEMICAL MASS TRANSPORT MEASUREMENTS TO DETERMINE FLEXIBLE MEMBRANE LINER (FML) LIFETIME

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ABSTRACT

The prediction of the service life of flexible membrane liners (FML) when exposed to chemicals has usually been by way of testing for physical or mechanical property changes after periodic exposure times. This paper presents an alternate approach by evaluating the chemical properties via four different diffusion related measurements of the exposed FML. These tests are water vapor transmission (WVT), radioactive tracer transmission (RT), water absorption (WA) and water vapor absorption (WVA).

This approach was tested for up to 6-month exposure times. The FMLs used were PVC, EPDM and CPE. The chemicals used were 10% NaOH (in water), 10% H_2SO_4 (in water), 10% phenol (in water) and 100% xylene. The WVT and RT were found to be quite reliable test methods, whereas the WA and WVA techniques experienced serious problems. The various transport coefficients showed all the expected types of behavior with chemical exposure:

- constancy with exposure (acids and bases on all FMLs)
- decrease with exposure (plasticizers leaching from FMLs)
- increase with exposure (Phenol-treated CPE and EPDM in H_2SO_4)

The WVT and RT results were generally comparable in all but one situation, namely the CPE-phenol case. Here WVT showed about a fourfold increase in permeability, whereas RT showed essentially no change in diffusivity.

The work reported here, even though it is preliminary, lends credence to the use of mass transport measurements to determine structural change in FMLs and hence a prediction of their lifetime.

INTRODUCTION

Before any meaningful design can begin on a FML-based liner or cover, its chemical compatibility with the contained materials must first be assured. Furthermore, this compatibility must be guaranteed for the lifetime of the facility. The challenge becomes quite formidable, however, when one deals with a complex waste stream from a landfill, i.e., leachate, or with waste streams from proposed facilities still in the design stage. The approach generally taken in such cases is to place the candidate FMLs in the actual or synthesized waste stream, expose them for a period of time, and then test them for changes in physical or mechanical properties. Depending on the magnitude of these changes, each particular geomembrane is then accepted or rejected. Problems invariably come up in these decisions, however, as to what are tolerable or intolerable changes in properties. Obviously, the ideal is no change. Yet, most FMLs do respond (even to water) with changes in thickness, volume, stiffness, modulus, elongation and strength. Often these changes are reasonably uniform with time.

PURPOSE

The present approach still uses an exposure of the FMLs to the proposed liquid, but now assesses the material's performance on the basis of its chemical properties - namely diffusion and permeability, instead of using mechanical and physical properties as the indicators.

The basic hypothesis is that if the FML has been degraded by the liquid to which it has been exposed, its internal structure will have been altered. If so, some form of diffusion properties (actually its mass transport characteristics) would probably be among the most sensitive parameters to evaluate. The trend in mass transport coefficient as a function of chemical exposure time could lead to a predictive method for FML lifetime determination. For example, it is well known in polymer science, that if the glass temperature is approached from the high temperature side (say from loss of plasticizer), the mass transport properties of the polymer can decrease significantly.

APPROACH

The following tests were evaluated and are discussed:

- water vapor transmission (WVT)
- radioactive tracer transmission (RT)
- water absorption (WA)
- water vapor absorption (WVA)

What is anticipated in these diffusion measurements is that abrupt or long term changes in the measured diffusion parameter will indicate unacceptable performance in a more clear-cut and unequivocal manner than physical or mechanical property testing. Stated differently, the structural changes, as measured by diffusion tests, might be more (or at least as) sensitive to chemical degradation of the FML as the structural changes, as measured by physical or mechanical tests.

Graphically, the anticipated behavior is as shown in the three alternates of Figure 1. Curve (a) shows an unwanted behavioral trend by having a breakthrough occur and a rapid increase in the measured diffusion property. Curve (b) illustrates the case where chemical degradation has occurred and the products of the reaction have "blocked" the microstructure resulting in an abrupt decrease in the measured diffusion property. (This could also be interpreted as a "stiffening" of the matrix.) Finally, in curve (c) the measured diffusion property remains constant showing an unaffected geomembrane diffusion to the liquid it was exposed to. Curve (c) is the preferred response.

Conditioning of Geomembranes

Conditioning of FMLs (exposed to waste) is vital to assessing their ultimate in-situ performance. The procedure selected should conform to the following requirements.

- representative of field conditions
- reproducible from one sample to another
- realistic to perform on a routine basis

Tub exposure is used here where the FML is folded into an empty tub which is used for its support and then the chemical liquid is placed within this FML lining.(8) Folds are indeed present, at 90° and 180°, as well as flat surfaces along the bottom and partially wet and dry surfaces along the sides. Most results given here will be for the flat samples. This procedure seems to closely simulate most of the FML lined field containment sites.

Three tub exposure setups have been constructed out of 4' x 8' plywood sheets (3/4" thick, exterior grade), each containing 12 tubs with FMLs in each. Three FML types (PVC, CPE and EPDM) were exposed to four chemicals (10% phenol, 10% hydrochloric acid and 10% sodium hydroxide - all in water solution, and 100% xylene). They were removed after short, medium and long exposure times (4 weeks, 12 weeks and 6 months) and when combined with the as-received data, give four data points to evaluate the aging process. The exposed FMLs were water-rinsed and air-dried after removal from the tubs. (Some were also vacuum-dessicated after this process to try to "dry" the samples completely). Next the individual specimens for diffusion testing were taken from the flat areas (no bend), the side portion (90° bend) and the corners (180° bend), although the majority of the work presented here is for the flat area samples. Test specimens were then stored in sealed plastic bags until testing commenced.

Methods of Measurement

a) Water Vapor Transmission

Water vapor transmission (WVT) tests were conducted according to the ASTM E-96 standard tests(1), with mechanical clamps being used for sealing these relatively thick (30 mil) FMLs. The test cups (containing water) were placed in an environmental chamber maintained at constant humidity and temperature. The cups were withdrawn at periodic intervals and weighed on a sensitive analytic balance. Calculation of WVT resulted from weight versus time according to the ASTM Standard(1).

b) Radioactive Tracer Transmission

The diffusion coefficient of a particular molecule can be determined if part of the molecule can be made radioactive. The diffusion coefficient is determined by monitoring the radioactive disintegrations versus time at some convenient point. We have used an energetic β-emitter, carbon-14 in benzene for the diffusing molecule. Figure 2 shows a schematic diagram of the test device. The diffusion coefficient is extracted from the counts versus time at the Geiger-Mueller tube face, using standard diffusion theory(5,6). The theory assumes one dimensional diffusion (i.e., a wide, thin sample).

c) Water Absorption

The diffusion coefficient for water in a particular FML can be determined by immersing the sample in water and monitoring the weight gain (i.e., its absorption) as a function of time. Diffusion theory is again used to obtain the diffusion coefficient(5,9). In most cases, however a final weight was not achieved making the analysis impossible.(10)

d) Water Vapor Absorption

In precisely the same manner as the water absorption test just described, the use of water vapor as an absorbed gas can be used to obtain the diffusion coefficient. The theory is exactly the same, and only a slight modification in the experimental setup is necessary. Now, instead of directly suspending an FML in a controlled temperature bath, it is encapsulated in a separate glass cylinder with water in the bottom which in turn is suspended in the water bath. Thus 100% humidity conditions are present, which allows for water vapor to enter the FML as a function of time. The increase in weight is plotted against time and diffusion theory used to extract the diffusion coefficient. However, the results we found are very suspect, since in most cases the initial part of the curve does not follow the $t^{\frac{1}{2}}$ law as required from diffusion theory.(10)

The details of all four test procedures (with actual data) can be found in Lord and Koerner(7).

PROBLEMS ENCOUNTERED

The major problem encountered during this study was the inability of the absorption methods (WA and WVA) to give consistently meaningful values for the diffusion coefficients. Running duplicate (and in many cases triplicate) samples does not mitigate this problem. This precludes the use of these methods to reliably monitor diffusion changes due to chemical exposure, even though very general (order of magnitude) values for the diffusion coefficients can be obtained, if enough samples are run.

In the water absorption method, for a great many cases, a final weight was not achieved even after 200 days of immersion. Without a final, equlibrium weight, the diffusion coefficient cannot be obtained.

In the water vapor absorption method, for a great many cases, the initial time dependence for absorption is nowhere near $t^{\frac{1}{2}}$, even though a constant weight is achieved in most cases. Thus a reliable diffusion coefficient cannot be obtained, because simple diffusion is not taking place.

Other problems encountered were minor compared to these.

RESULTS

The results of each test method on each FML will be discussed separately. Table 1 gives a summary of all results. In Table 1 is given the percentage change after six month exposure, plus the trend occurring at the six month time (increasing, decreasing or steady). The criteria for effect of the chemical on the FML is twofold:

- the absolute change in mass transport property
- the trend (time variation) in the mass transport property.

Both criteria must be fulfilled for a deleterious effect to have occurred, namely a large absolute increase and a significant rate of increase (with exposure time) at 6 month's exposure.

a) WVT Results

The results of chemical exposure are shown in Figures 3, 4 and 5. The results are the average of two runs, which never differed from each other by more than 5%. In Figure 3, it is seen that the NaOH and H_2SO_4 solutions (10%) have very little effect on the WVT of PVC, even for periods of one half year. On the other hand, the chemicals which "leach" plasticizer from PVC, namely phenol and xylene, produce a very definite lowering of the WVT. PVC is noticeably stiffer after exposure to these two chemicals. It is very well known in the polymer field that if a materials becomes stiffer, the transport properties usually decrease in magnitude. This result lends much credence to the use of simple transport measurements to monitor the structure of the FMLs.

Figure 4 shows that the WVT in EPDM is affected very little by 6 months' exposure to the four chemicals.

Figure 5 shows that the WVT in CPE is affected very little by acids and bases, but significantly increases upon exposure to phenol. CPE is completely destroyed by xylene in about a day, so no WVT results are shown for this combination.

Thus it is seen that the WVT can be significantly reduced (Fig. 3), remain the same (Fig. 4) or increase significantly (Fig. 5) upon chemical exposure. The most interesting result would seem to be the effect of phenol on CPE, for if the WVT trend continues upward, a time will be reached very soon where the WVT would reach an unacceptably high value for containment of phenol-type wastes.

b) Radioactive Tracer Measurements

Figures 6, 7 and 8 show the results of the diffusion coefficient for benzene as a function of chemical exposure. Duplicate samples were run in many cases to check the reproducibility; it was always better than 5%. The tracer results for PVC (Fig. 6) are in general agreement with those found in the WVT work in PVC (Fig. 3). The exposure to acid and base do little to the diffusion coefficient, whereas the leaching agents, phenol and xylene lower the diffusion rate as the plasticizer is leached-out and the PVC becomes stiffer.

The results for EPDM (Fig. 7) are in exact agreement (with respect to relative change) with those given by WVT (Fig. 4). This agreement between the two methods for PVC and EPDM is very encouraging, and again gives confidence in the general mass transport measurement approach to monitor structural changes in the FMLs.

The CPE diffusion results, as a function of chemical exposure, are shown in Figure 8. Agreement with the WVT results for CPE (Fig. 5) is good for the acid and base, but is very poor for phenol. Whereas the WVT results showed CPE to become approximately four times more permeable after 6 months exposure to phenol, the benzene diffusion rate shows essentially no increase after 6 month's exposure to phenol. All the phenol results on Figure 8 were duplicated within agreement of 5%. This is quite an important (and possibly discouraging) result, because the CPE/phenol situation appeared from the WVT to be a prime candidate for FML lifetime prediction via transport measurements.

c) Water Absorption and Waste Vapor Absorption

As mentioned in the previous section, there are significant problems in determining the diffusion coefficients in FMLs via water absorption or water vapor absorption. The main problem with liquid water absorption is that in many cases the FML does not achieve a constant (equilibrium) weight during a very long immersion time. Many samples were monitored for 200 days with no equilibrium weight achieved. The diffusion coefficient cannot be determined without this final weight. In spite of this serious problem, many samples showed very nearly the expected (Fickian)

$t^{\frac{1}{2}}$ power law in the initial stages of absorption.

In the case of water vapor absorption, the main problem is the lack of an initial $t^{\frac{1}{2}}$ time dependence, even though a constant weight is achieved in a reasonable time. The diffusion is not simple Fickian and hence calculation of a diffusion value from the data is extremely suspect.

There are other problems in this measurement technique:

- PVC absorbs a very small amount of water, making analysis next to impossible.
- Some samples actually lose weight while in contact with water, indicating some of the chemical gained during exposure is still coming out of the FML (this even happened in some vacuum-dessicated samples).
- In many cases the data is scattered and shows no definite trends.

The conclusion is that water and water vapor absorption are not reliable methods with which to determine the diffusion coefficient of water in commercial FMLs. Some others(4,2,3) who has worked in this area of absorption measurement have obtained the same type of results (and problems) as those found here, however their conclusions were not as negative, with respect to the use of the absorption method, as the present authors feel is warranted.

On the positive side, when the method does work, it shows diffusion coefficients consistently in the 10^{-9} cm^2/sec range for both the thermoplastic (CPE and PVC) and thermoset (EPDM) types. However, it is not possible to systematically follow the degradation of FMLs via these absorption type measurements. There are simply too many problems with this technique as applied to these commercial materials.

CONCLUSIONS

The first goal of the present work was to ascertain if mass transport measurements could be used reliably to determine structural changes in FMLs. If this was the case, then the second goal was to determine if systematic trends developed in the mass transport values as a function of exposure to various liquid wastes. If present, these trends would be used (the final goal) to determine, probably by long term extrapolation of the data, the lifetime of the FML in a particular waste.

Using a matrix of 3 FMLs and 4 chemicals, the first two goals were definitely realized. Two of the four mass transport techniques used (WVT and RT) gave very reproducible and apparently quite reliable indication of changes observed from the trends in the data. Furthermore in all but one case the two methods agreed completely as to the relative changes. The well known resistance of most FMLs to acids and bases has been amply demonstrated. The mechanical stiffening of PVC due to loss of plasticizer has been demonstrated via transport measurements. In metallic and polymeric systems mass transport properties have been found to be at least as sensitive (and in some cases more sensitive) to structural change as mechanical and physical properties.

The most interesting situation as far as lifetime prediction is concerned is the CPE-phenol case. Here a very significant (about 4-fold) increase in the WVT takes place with 6 month's exposure. If this significant increase continues with exposure time, this would be an ideal case in which to determine a limiting upper value of WVT, above which the FML would become unuseable for containment. At this juncture the RT results do not corroborate the WVT for the CPE-phenol case, there being essentially no change in the diffusion coefficient after 6 month's exposure.

Thus the indicated significant increase in WVT for CPE exposed to phenol must be investigated in greater detail, to ascertain if it is a true effect, or if the lack of an effect, as indicated in the RT measurements, is closer to the true structural effect. This situation should be run again for complete verification of the data, and possibly other experimental techniques (e.g., microstructural observations, glass temperature measurements, etc.) should be brought to bear on this particular FML/chemical situation. Other systems with similar chemistry should also be investigated.

It is felt that the goals of the study were achieved and a very definite possibility exists for determining the lifetime of FMLs through mass transport measurements on chemically-exposed samples.

ACKNOWLEDGEMENTS

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TABLE 1. SUMMARY OF THE WVT AND RT RESULTS FOR 6 MONTH EXPOSURE

	6 Month's Exposure	
PVC	WVT	RT
H_2SO_4	+ 12% - steady	+ 4% - steady
NaOH	+ 10% - steady	-11% - steady
Phenol	- 30% - steady	-50% - steady
Xylene	- 25% - steady	-85% - steady
EPDM		
H_2SO_4	+138% - rising	+37% - rising
NaOH	+100% - steady	+20% - steady
Phenol	+ 70% - steady	+17% - steady
Xylene	-	-
CPE		
H_2SO_4	+ 54% - steady	+15% - rising
NaOH	+ 46% - steady	-11% - steady
Phenol	+260% - rising	- 9% - steady
Xylene	-	-

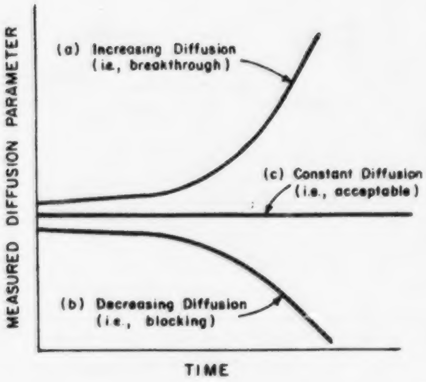

FIG. 1. Various Possible Trends in Diffusion Properties as a Result of FML Exposure Time

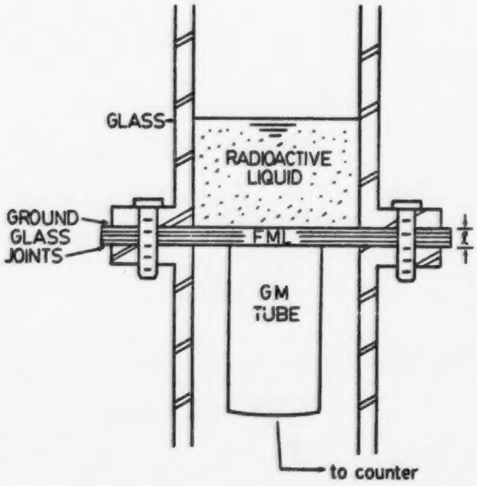
GLASS
RADIOACTIVE LIQUID
GROUND GLASS JOINTS
FML
x
GM TUBE
to counter

FIG. 2. Schematic Diagram for Determining Diffusion Constant Using Radioactive Tracers

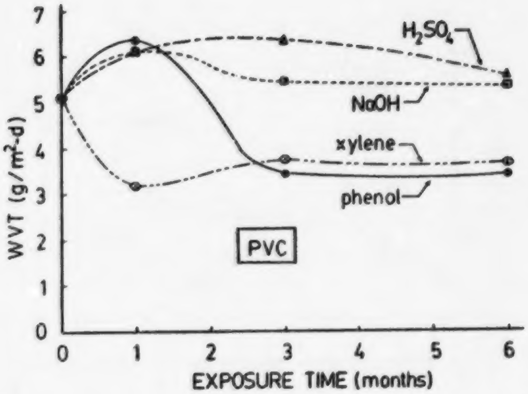

FIG. 3. WVT Results for PVC as a Function of Exposure Time to Various Chemicals

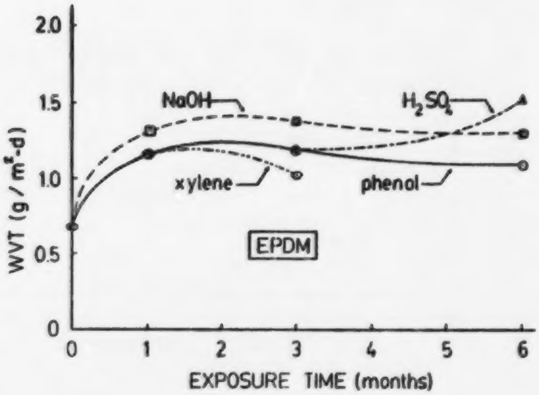

FIG. 4. WVT Results for EPDM as a Function of Exposure Time to Various Chemicals

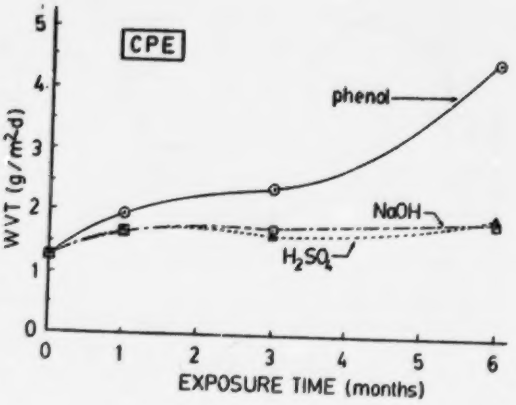

FIG. 5. WVT Results for CPE as a Function of Exposure Time to Various Chemicals

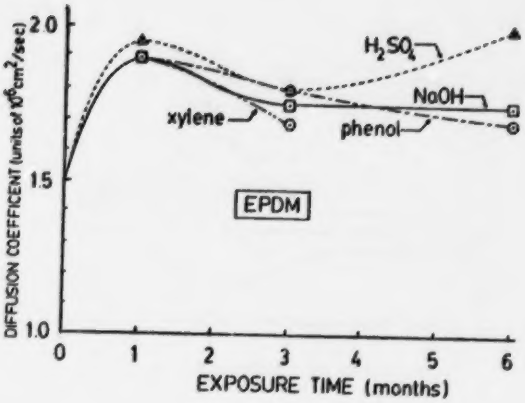

FIG. 7. Diffusion Coefficient of Radioactively Tagged Benzene (^{14}C) in EPDM as a Function of Exposure Time to Various Chemicals

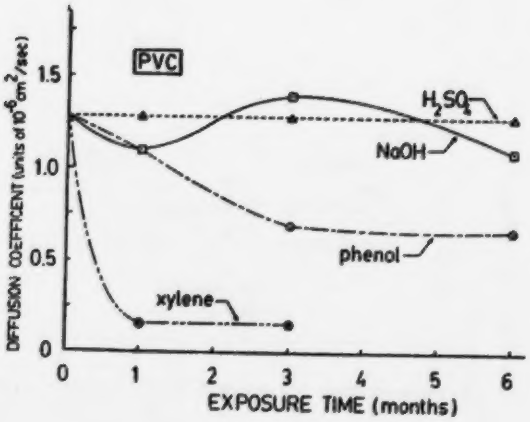

FIG. 6. Diffusion Coefficient of Radioactively Tagged Benzene (^{14}C) in PVC as a Function of Exposure Time to Various Chemicals

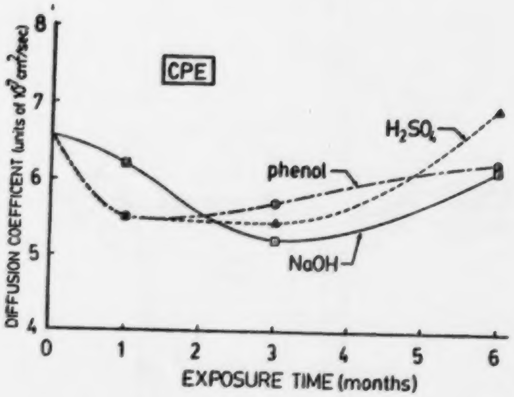

FIG. 8. Diffusion Coefficient of Radioactively Tagged Benzene (^{14}C) in CPE as a Function of Exposure Time to Various Chemicals

DEVELOPMENT OF A TECHNIQUE FOR RETROFITTING IMPOUNDMENTS WITH GEOMEMBRANES

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ABSTRACT

A technique has been tested for retrofitting in-service liquid impoundments with a flexible membrane liner. Tests of the pull-through technique under simulated field conditions were implemented at a 0.4 hectare (ha) (1 acre) impoundment test facility.

The liner material selected for these tests was 0.91 millimeter (mm) chlorosulfonated polyethylene, which was constructed of two 0.4 mm layers with reinforcing scrim between them. A specially-designed steel bar, a self-adjusting rope bridle, and a tow vehicle were used to pull the sheets of liner material along the contour of the water-filled impoundment.

These tests demonstrated the feasibility of using the technique to install large sheets of liner material. The size and shape of the impoundment, liner material, and weather conditions are factors that influence the successful application of the technique. The test set-up and results are described in this paper.

INTRODUCTION

This paper presents the results of a project to develop liner retrofit techniques and to demonstrate one or more methods under large-scale conditions. Three in-service retrofit techniques were evaluated during the project. These techniques were termed pull-through, pump-over, and float-and-sink. The results of the conceptual and model studies have been previously reported (1), and will not be included here. The pull-through technique was selected for large-scale tests.

RESEARCH APPROACH AND FIELD ACTIVITIES

Large-Scale Study

The test impoundment could accommodate liquid to a depth of 2 meters (m). This facility was previously used as a test impoundment that required the conventional installation of a geomembrane. Tests of the retrofit technique were conducted with this previously installed liner in place.

Approach for Single Sheet Tests

Single sheet tests of the pull-through technique were performed using one sheet of liner material approximately 12.2 m by 64 m in size. The material selected was 0.91 mm chlorosulfonated polyethylene, which is constructed of two 0.4-mm layers with reinforcing scrim between them. This material was selected because of its high tensile strength, availability, and its prevalent use in the liner industry.

Figure 1 shows a conceptual drawing of the single sheet test setup. Two steel plates, eyebolts, a self-adjusting rope bridle, and a tow vehicle were used to pull the sheet lengthwise down the side slopes, across the bottom, and up the opposite slope. The leading edge of the sheet was positioned at the impoundment berm. Two 6.1-m sections of flat steel plate hinged together in the center were attached to the edge of the sheet by wrapping the sheet around the bar. The bar was equipped with two nuts welded along the top of the bar at

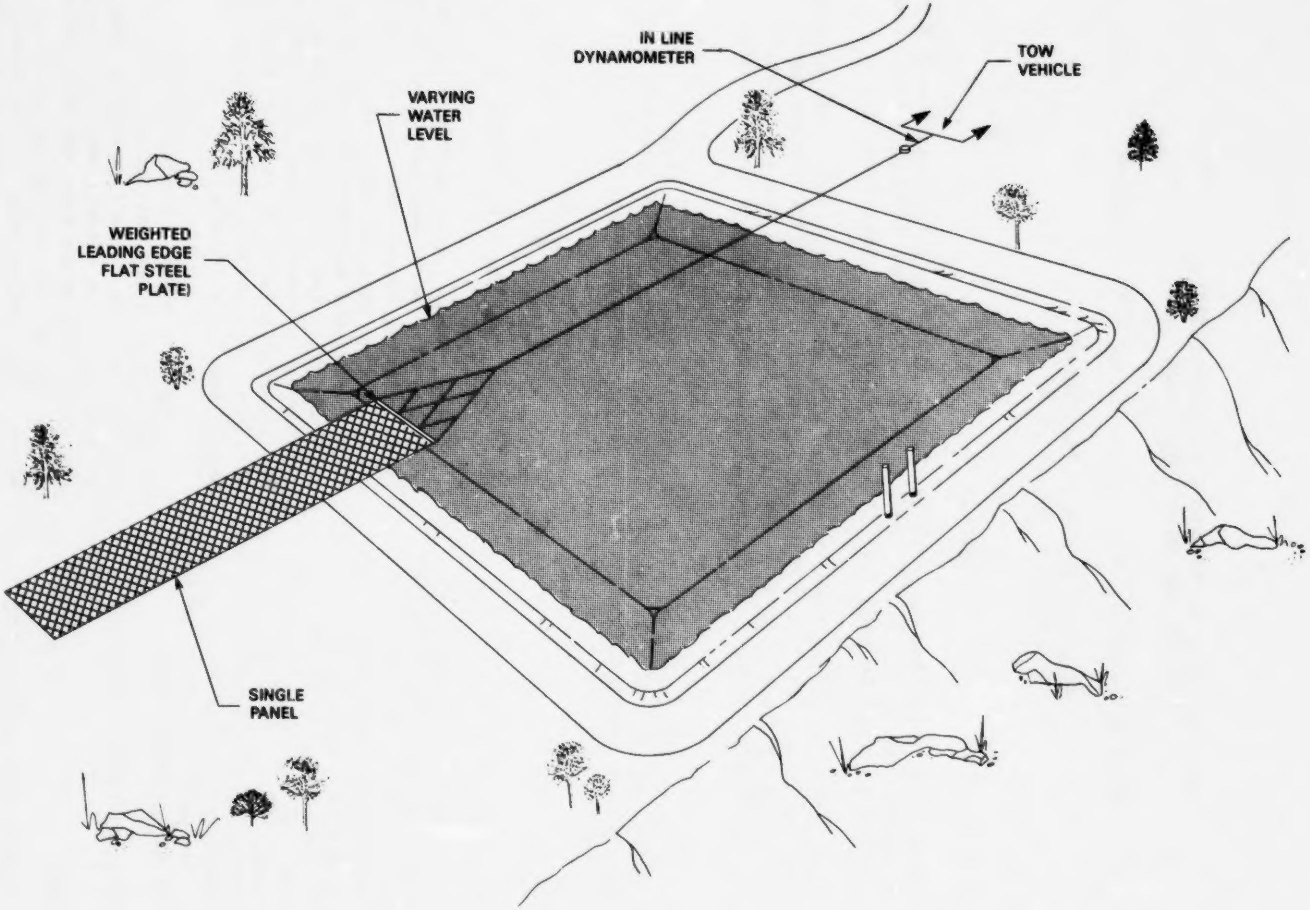

IN LINE DYNAMOMETER
TOW VEHICLE
VARYING WATER LEVEL
WEIGHTED LEADING EDGE FLAT STEEL PLATE)
SINGLE PANEL

Figure 1. Conceptual drawing of the single sheet pull-through test setup.

approximately 2-m intervals. After wrapping the liner material around the bar, holes were cut through the liner at each attachment location and eyebolts were then screwed into the nuts. A self-adjusting bridle made of 12-mm polyethylene rope was attached to the eyebolts. The rope bridle was comprised of short segments of rope connected by shackles. The shackles allowed the segments of rope to adjust position under tension. This design was chosen to distribute the pulling load as evenly as possible along the bar. The other end of the bridle was attached to a tow vehicle located on the opposite side of the impoundment. A dynamometer was placed in-line between the bridle and the vehicle to measure the tension in the line.

Four tests were conducted using the single sheet and two-section steel bar. In three of the tests, the sheet was pulled down the side slopes and across the bottom of the impoundment. The objectives of these tests were to determine: (1) the forces required to pull the sheet and steel plates across the impoundment; (2) how well the steel bar distributed the pulling forces along the sheet; and (3) whether or not the weighted leading edge would keep the membrane on the bottom of the pond.

The fourth trial involved pulling the panel along the sloping side of the impoundment, keeping the center of the bar over the toe of the side slope. In this configuration, half of the panel was lying at at angle on the side slope, partially out of the water. The other half of the panel was lying flat on the bottom of the pond. The objectives of this test were to determine: (1) whether or not the panel could be pulled along the berm without tearing; and (2) whether or not the bar would bend at the hinge and therefore conform to the change in slope angle at the toe.

To accomplish the pull along the toe of the berm, the sheet was moved into position so that its center and hinge of the bar would be in line with the toe. The bar was then moved into position. The bar was attached to the sheet in the same manner as before. The sheet was then pulled along the side slope to the opposite side of the impoundment.

Single Sheet Test Results

The maximum pulling forces for each test as measured with the dynamometer were as follows:

(1) Test 1 - 8,900 newton (N) (2000 lb-force)
(2) Test 2 - 7,600 N (1700 lb-force)
(3) Test 3 - 9,800 N (2200 lb-force)
(4) Test 4 - 6,200 N (1400 lb-force)

These values occurred when the bar was pulled up the side slope. The force required to pull the sheet along the berm (Test 4) is the smallest of the four results because approximately one half of the sheet was sliding over the dry surface of the existing liner where the friction was less than that under water. For an earthen impoundment, the friction along the dry soil berm would likely be higher than the wet subgrade below water level. Tests of the tensile strength of the sheet under field conditions were not performed, since the minimum strength of the sheet at similar temperatures was available from the manufacturer.

Approach for Double Sheet Test

A second large-scale test was performed to determine the feasibility of pulling wider sheets of liner material. For this experiment, the liner material previously used for the single sheet tests was repositioned to be pulled broadside into the test impoundment. A longer bar similar to that used previously was attached to the long edge of the sheet. The sheet was then pulled down the slope until the trailing edge reached the top of the berm. A second sheet of the same size was then seamed to the first sheet and both sheets were pulled into the water until the second sheet covered the side slope. Figure 2 shows a conceptual drawing of this experimental setup. The first sheet is shown pulled into the water and along the bottom of the impoundment. The second sheet is shown bonded to the first sheet. The entire double sheet is pulled along the contour of the impoundment by the weighted leading edge. Three ropes were used to attach the bar and bridle to the tow vehicle.

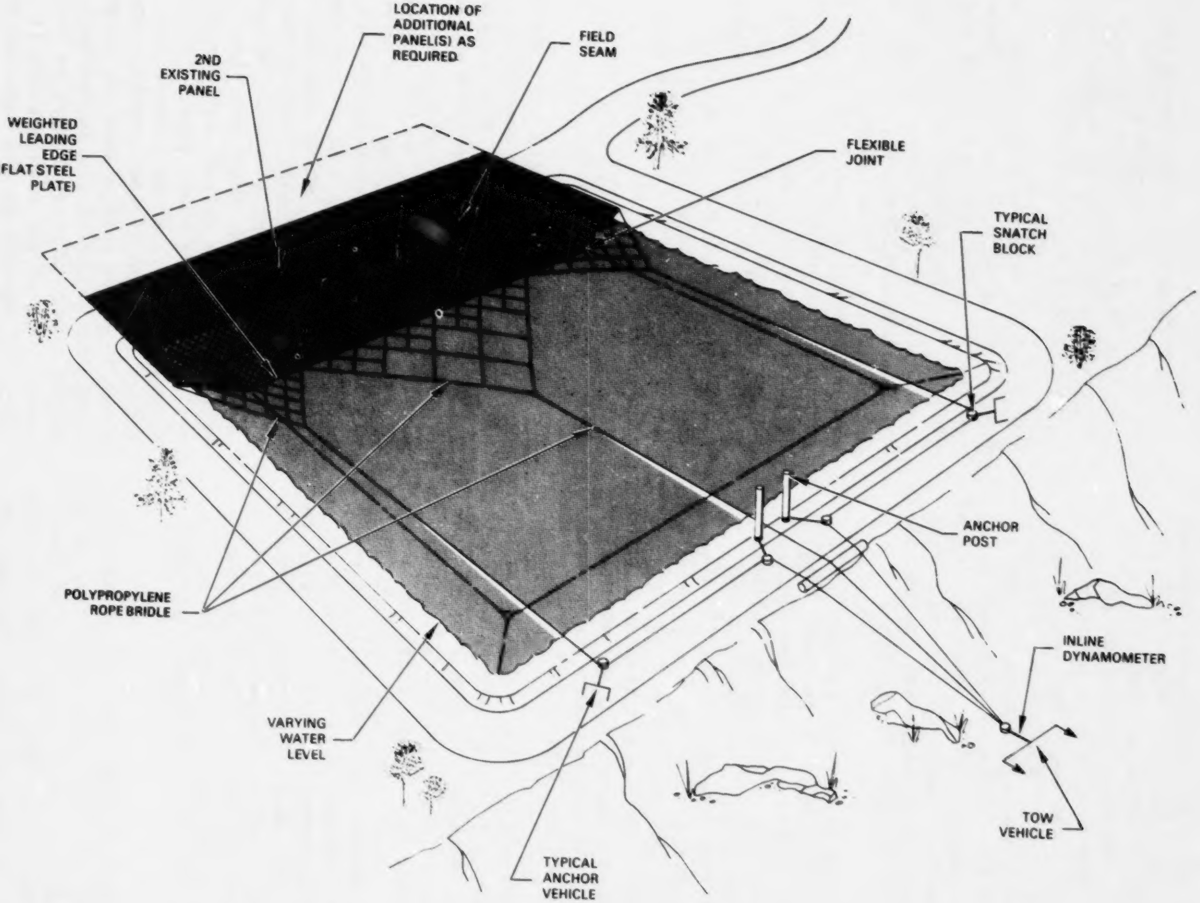

LOCATION OF ADDITIONAL PANEL(S) AS REQUIRED
FIELD SEAM
2ND EXISTING PANEL
WEIGHTED LEADING EDGE (FLAT STEEL PLATE)
FLEXIBLE JOINT
TYPICAL SNATCH BLOCK
ANCHOR POST
POLYPROPYLENE ROPE BRIDLE
INLINE DYNAMOMETER
VARYING WATER LEVEL
TOW VEHICLE
TYPICAL ANCHOR VEHICLE

Figure 2. Conceptual drawing of the double sheet pull-through test setup.

The bar used for this experiment consisted of three rigid steel bar sections joined with two hinged joints. The details of the bar are illustrated in Figure 3. The two flexible joints were designed to allow the bar to follow the contour of the impoundment. The flat bar cross-section was selected to provide minimum rotational stress on the liner material as it was pulled along the bottom of the impoundment.

The bridles used for this test were similar in design to that used for the single sheet tests. However, they contained many more rope segments due to the additional width of the leading edge and the increased weight. One large bridle was attached to the eyebolts along the 45.7-m rigid center section of the bar. Two smaller bridles were attached to the 9.14-m end sections of the bar so that the centers of the bridles were in line with the flexible joints. Shackles were used to attach the bridles to the bar and to hold the bridle rope segments together in a triangular pattern. Tow lines made of the same rope were then attached to each of the three bridles. The rope was rated at a breaking strength of 22,300 N (5000 lb-force). Each line was run across the surface of the test impoundment. This setup was shown previously in Figure 2. The two outside lines were rigged through snatch blocks to the tow vehicle. The center line was run directly across the impoundment to the tow vehicle.

Double Sheet Test Results

To begin the test, the slack was taken out of the bridle and the bar was pulled forward to straighten out the liner. Pulling was stopped to inspect the sheet. A tear was discovered just behind one of the flexible joints. This failure occurred just behind a factory seam. Possible causes were: (1) unequal loading on the sheet from the bar; (2) a defect in the sheet as a result of manufacturing; or (3) a small puncture or tear introduced in previous tests using the same material. There were no other failures anywhere along the sheet. To remedy this failure, the bridle was removed from the bar and the sheet was then wrapped around the bar past the failure point. The bridle was reconnected and the test pull was restarted.

The first sheet was pulled over the berm of the existing liner and down the side slope. The first sheet was pulled into the water until the trailing edge reached the top of the berm. At that position, the tow vehicle was stopped to allow the second sheet to be attached. The second sheet, measuring 12.2 by 64 m, was fan-folded and positioned along the first sheet, maintaining a 0.15-m overlap with the first sheet. The sheets were seamed together using a bodied solvent adhesive recommended by the manufacturer. The seam was allowed to cure for two hours before being pulled. According to one manufacturer of chlorosulfonated polyethylene, the seam should have cured to approximately 60 percent of the final cure strength in two hours. Since no failures occurred at this seam, the bond strength was strong enough to withstand the tensile loading of the second sheet.

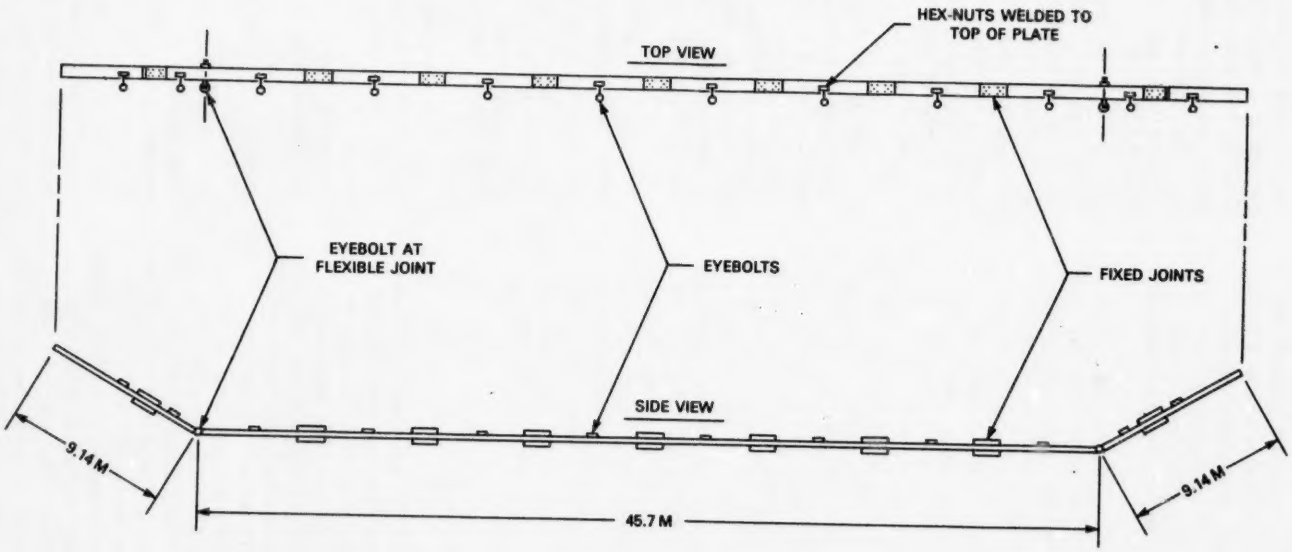
With the two sheets bonded together, there were approximately 1,560 m^2 of sheet weighing 1,907 kilograms (kg). The bar weighed approximately 7.55 kg per linear meter, or 483 kg. The total mass of the bar and sheet was approximately 2,400 kg including hardware and the plates used at each fixed joint.

After the pull was completed, the sheet was examined for damage. One tear was found, located approximately 0.3 m behind the flexible joint. The tear was similar to the first tear, occurring at a factory seam. It was larger than the first tear, approximately 0.3 m in length.

The maximum force required to pull both sheets was approximately 22,200 N (4990 lb-force), which occurred at the start of the test. After the starting friction was overcome, the force dropped to approximately 13,300 N (2990 lb-force).

Stress Loads on Liner Material

The tensile load on the liner material during the single and double sheet tests can be estimated to provide a basis for evaluation of the technique and its application. During the single and double sheet tests, loadings on the liner material were not measured directly. Direct measurement would have required the use of strain gauges attached to the sheet, or some similar technique. This effort was beyond the scope of the project. However, these loadings can be estimated by assuming that the loads measured by the dynamometer



WEIGHTED LEADING EDGE

HEX-NUTS WELDED TO TOP OF PLATE
TOP VIEW
EYEBOLT AT FLEXIBLE JOINT
EYEBOLTS
FIXED JOINTS
SIDE VIEW
9.14 M
45.7 M
9.14 M

Figure 3. Details of the weighted leading edge bar used during the double sheet test.

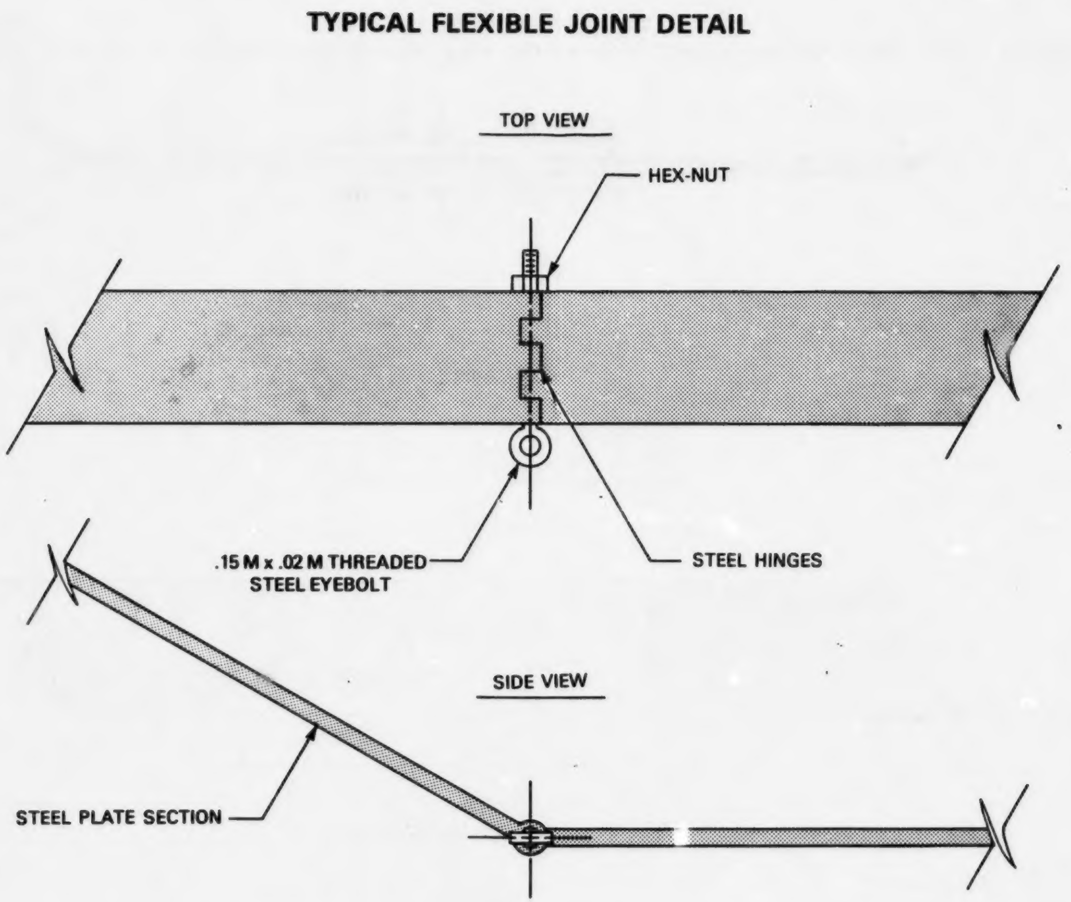

TYPICAL FLEXIBLE JOINT DETAIL
TOP VIEW
HEX-NUT
.15 M x .02 M THREADED STEEL EYEBOLT
STEEL HINGES
SIDE VIEW
STEEL PLATE SECTION

Figure 3. Details of the weighted leading edge bar used during the double sheet test (Cont'd).

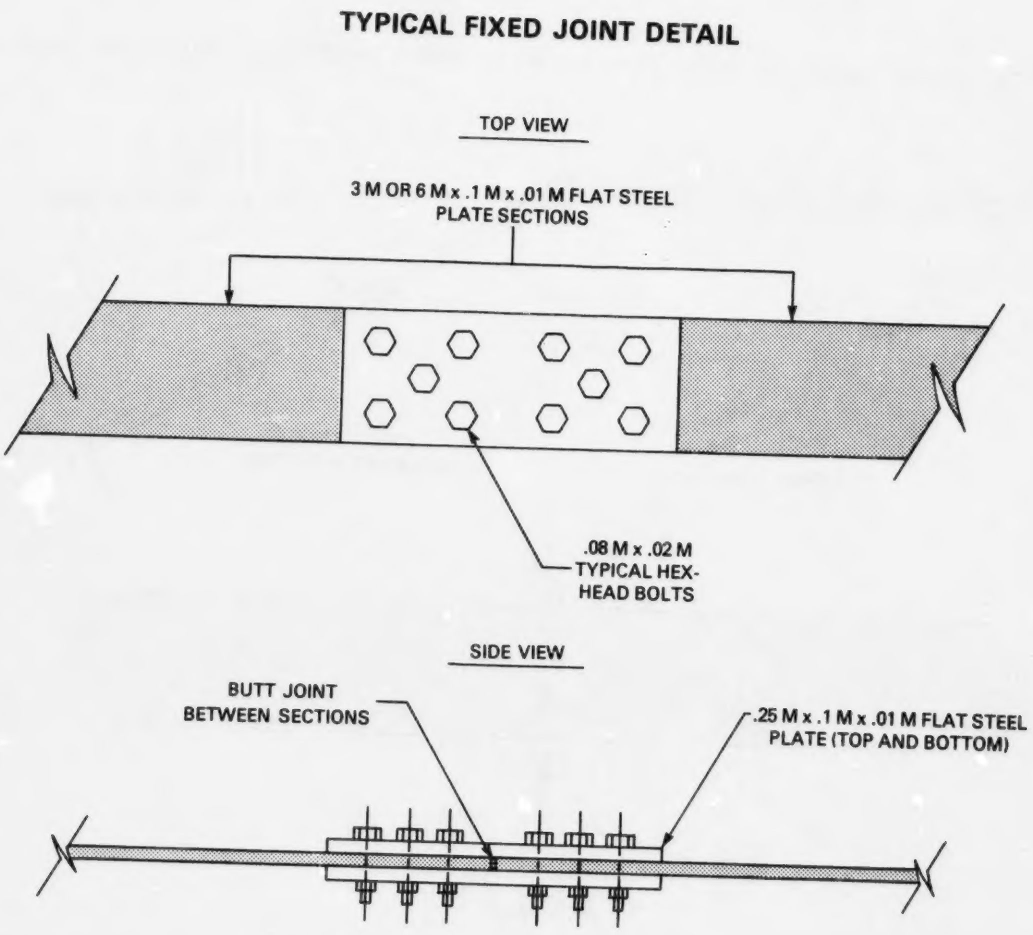

TYPICAL FIXED JOINT DETAIL
TOP VIEW
3 M OR 6 M x .1 M x .01 M FLAT STEEL PLATE SECTIONS
.08 M x .02 M TYPICAL HEX-HEAD BOLTS
SIDE VIEW
BUTT JOINT BETWEEN SECTIONS
.25 M x .1 M x .01 M FLAT STEEL PLATE (TOP AND BOTTOM)

Figure 3. Details of the weighted leading edge bar used during the double sheet test (Cont'd).

were uniformly applied entirely to the liner material. This represents a worst case analysis, because some percentage of the dynamometer reading is attributable to the weight of the bar and frictional forces in the rope bridle system. These forces are not imposed on the liner. For the worst case, the loadings measured by the dynamometer can be assumed to approximate the average forces delivered to the liner immediately behind the steel bar attached to the material during each test pull.

For the single sheet tests, the assumptions and data needed to estimate liner tension loadings are as follows:

(1) All forces are equally distributed along the length of the sheet immediately behind the bar;

(2) The length of the sheet is 12.2 m; and

(3) The maximum force measured during the four separate single sheet tests is 9,800 N.

The tension per linear centimeter of liner material is calculated as follows:

$$T_1 = \frac{9{,}800\ \text{N}}{12.2\ \text{m}} \times \frac{1\ \text{m}}{100\ \text{cm}} = 8.03\ \text{N/cm}$$

For the double sheet tests, the assumption of equal loading behind the bar is not completely valid, since a small tear occurred at one point. However, to derive a load comparable with the single sheet load calculated above, the tension per linear centimeter for the double sheet test is:

$$T_2 = \frac{22{,}200\ \text{N}}{64\ \text{m}} \times \frac{1\ \text{m}}{100\ \text{cm}} = 3.47\ \text{N/cm}$$

A representative of the manufacturer of the liner material indicated that there are two factors which will influence the loading that the material can withstand (A. Peterson, J.P. Stevens Co., personal communication, 1984). These are: (1) the temperature of the sheet; and (2) the length of time the material is in tension. The increased temperature softens the elastomer, reducing the tensile strength. The longer the material is in tension, the more the material will exhibit slow elongation. This implies that a relationship exists between the strength of the liner material and the rate that it is loaded in tension. The manufacturer's data for tensile strength of the material indicate that the scrim by itself can withstand 131 N/cm tensile loading at approximately 49°C. The test method used to measure this value specifies a rate of pull, or length of time the material is in tension.

Calculation of Theoretical Sheet Size

Given the calculations from the field test data and the manufacturer's data, it is possible to evaluate the loadings experienced in the field and to calculate the theoretical sheet size that could be pulled using the technique.

During the single sheet tests, no liner failures occurred. The calculated maximum uniform tensile load of 8.03 N/cm is well below the expected maximum strength of 131 N/cm. During this test, the bar was rigid and flat across its length. The force on the bar was probably distributed equally to the liner. The sheet temperature when the maximum loading occurred was approximately 32°C. Thus, the strength of the sheet was above 131 N/cm, corresponding to a sheet temperature of approximately 49°C. The bar was pulled at a constant rate across the pond and stopped at the opposite berm. The maximum dynamometer reading occurred when the bar was pulled up the opposite berm. The highest tension in the sheet occurred just before it began to move up the slope.

The successful application of the pull-through technique for retrofitting in-service impoundments will depend upon many factors. The most important factors are as follows:

(1) The tensile strength of the liner material under the prevailing site conditions;

(2) The variation in liner material strength due to sheet temperature;

(3) The uniform application of pulling forces along the entire length of the bar;

(4) The geometry of the impoundment;

(5) The conditions of the bottom and side slopes of the impoundment; and

(6) Wind and rain conditions during installation of the liner.

The tensile strength of the liner material is the most critical technical factor associated with successfully applying the pull-through technique to any size impoundment facility. The material must withstand the mechanical stresses from the bar as well as any loading due to the weight of solids or liquids in the impoundment. The mechanical aspects of pulling the sheet will not limit the size of facility that can be retrofitted. Any number of winches can be operated together to move the bar and liner at a constant rate.

The slope of the embankments will influence the application of the technique. The steeper the slope, the more force will be required to pull the bar up the slope and over the berm. The conditions of the bottom and side slopes of the impoundment are also important factors. Debris on the bottom of the impoundment may tear the liner or prevent the bar from moving properly along the bottom. Unstable, soft subgrade will not adequately support the liner material after retrofit. All of these factors must be considered before initiating a retrofit project using the pull-through technique. As a minimum, the bar should be initially pulled across the impoundment without the liner to help identify any foreign objects that may damage the liner or prevent the retrofit from being completed. The proper liner material must also be selected, based upon the required mechanical properties for successful installation.

The theoretical limits of the pull-through technique with respect to impoundment size can be computed assuming: (1) equal loading on the sheet; (2) pulling forces equal the weight of the sheet; and (3) constant pulling rate along the bar. The following example calculation will illustrate the computation for a rectangular impoundment:

Given:

(1) Rectangular impoundment with ends 76.2 m in width;

(2) 0.91-mm chlorosulfonated polyethylene liner material with minimum tensile strength of 131 N/cm;

(3) Installation temperature of 21°C;

(4) Side slopes no steeper than 3 to 1; and

(5) Mass of liner 1.2 kg/m^2.

Find:

(1) The length and surface area of the impoundment that can be theoretically retrofitted using the pull-through technique.

Solution:

(1) Compute the total tensile force that can be sustained by 76.2 m of liner material:

$$\text{Force} = 131\ \text{N/cm} \times 76.2\ \text{m} \times 100\ \text{cm/m}$$

$$= 998{,}220\ \text{N}$$

(2) Compute the area of liner material equal to the load from (1) above:

$$\text{Area} = \frac{998{,}220\ \text{N}}{1.2\ \text{kg/m}^2}$$

$$= 84{,}883\ \text{m}^2$$

(3) Compute the length and surface area:

$$\text{Length} = \frac{84{,}883\ \text{m}^2}{76.2\ \text{m}}$$

$$= 1{,}114\ \text{m}$$

$$\text{Surface Area} = \frac{84{,}883\ \text{m}^2}{10{,}000\ \text{m}^2/\text{ha}}$$

$$= 8.49\ \text{ha}$$

This calculation suggests that, given the assumptions listed above, a sheet of liner material measuring 76.2 m wide by 1,114 m long would be strong enough to withstand pulling from one end. A piece of material this size would be sufficient to retrofit an impoundment having a surface area of nearly 8.5 ha.

This calculation takes into consideration only the specified maximum tensile strength of the material at 49°C, and does not include a factor of safety. Other factors, such as the effect of sheet temperature on strength, the uniform application of pulling forces, the impoundment geometry, and the condition of the bottom and side slopes of the impoundment, must be considered to estimate the limits of this retrofit technique.

SUMMARY

The bar and bridle performed as intended for the single-sheet tests. The bar conformed to the contour of the impoundment, keeping the sheet below the liquid level. Bending of the bar occurred because of variations in stretch of the polypropylene rope and uneven slack in the lines. This problem would be remedied by attaching additional lines to the bar to distribute the pulling forces more evenly along the leading edge.

The large-scale tests demonstrated the feasibility of the pull-through technique for moving large sheets of liner material along the contour of the test impoundment. The double sheet, measuring 64 m wide by 24.4 m deep, was pulled a distance of approximately 24.4 m. The force required to initially move the sheet and bar was approximately equal to the force of gravity on the sheet and bar. The force required to keep the sheet moving was approximately 13,300 N.

Weather conditions during the pull-through test were unavoidably hot (ambient temperature of approximately 38°C) and sunny. These conditions caused heating of the sheet, reducing its tensile strength. The coefficient of friction between the sheet and the ground was large because at elevated temperature the sheet conforms more closely to the irregularities of the ground. These factors probably contributed to the two sheet failures during the double sheet test. Materials similar to chlorosulfonated polyethylene, such as polyvinyl chloride and chlorinated polyethylene, will also be softened under similar conditions. Whenever these materials are used, the application of this technique should be limited to cooler temperature conditions.

The method used to attach the liner material to the bar was satisfactory for this demonstration. The sheet did not unwrap from the bar at any point.

Estimation of the stress loading on the liner material during the tests approximate the weight of the material itself. The stress loading per unit length of bar thus depends on the length of the bar. For example, the stress loading for the single sheet test was 8.03 N/cm of bar width, whereas it was 3.47 N/cm of bar width for the double sheet test. A retrofit application project can be designed to take advantage of this fact so as not to exceed the strength of the liner material chosen.

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THE ROLE OF IMPOUNDMENT SIZE IN CONTROLLING ENVIRONMENTAL RISK AT HAZARDOUS WASTE SURFACE IMPOUNDMENTS

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ABSTRACT

The environmental damage due to a hazardous waste surface impoundment failure is greatly affected by the impoundment size (volume). This research determined if multiple-small impoundments, designed to contain the same waste volume as one large impoundment, would present less environmental risk than the large impoundment. Three surface impoundment design scenarios (large, small-attached, small-unattached) were compared based on the environmental damage that would result from two failure situations: surface (catastrophic) and subsurface (long-term). The reduced risk from using smaller impoundments was then compared to the increase in construction cost due to constructing more than one impoundment. It was concluded that there are no real advantages to limiting the size of surface impoundments. Several design features are suggested which should greatly reduce the risks associated with surface impoundment failures.

INTRODUCTION

Basis for optimizing the design of surface impoundments (SI) should invlude consideration of the following: cost, geotechnical and hydrogeologic factors, land use, engineering design, environmental performance and total waste volume. Since many of these concerns are interrelated, optimization SI design can be a complex task. To help simplify the task of SI design optimization, this paper addresses the following more general question:

> "For a given desired SI volume, what are the costs and environmental risks associated with using one large SI instead of multiple small SI's."

An extensive literature review and inquiries directed to SI experts indicated there had been little previous study of this question. Efforts to address the question focused on the following aspects of surface impoundments:

1) Design Scenarios;
2) Construction Costs; and
3) Surface and Subsurface SI Failure.

Mechanisms of failure are examined along with the risk associated with each type of surface failure for several impoundment configurations (Figure 1).

IMPOUNDMENT DESIGN SCENARIO SELECTED FOR STUDY

An EPA study (1983) found that almost 85% of industrial SI's are under 4 acres (1.6 hectares) in surface area (Figure 2). Consequently, the 1.6 hectare was selected to represent a single large SI. To facilitate comparison, four impoundments were selected to represent multiple small SI's. Because multiple small SI's may be constructed with or without common dikes (Figure 1) and because shared dikes may increase the risk of multiple SI failures (discussed later in this paper), both of these were considered in this study. In each case small SI's were assumed to contain 1/4 the volume of the one large SI, and all SI's were assumed to have been constructed by the excavate and fill method (ie., partially above and partially below grade).

For an unbiased comparison of the three configurations (one large, multiple small

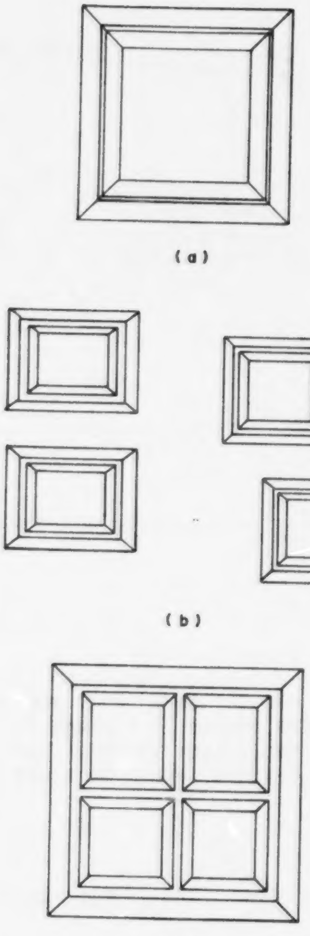
(a)
(b)
(c)

FIGURE 1. THE THREE SI CONFIGURATIONS CONSIDERED
(a)LARGE SI (b)4 SMALL INDEPENDENT SI
(c)4 SMALL SI JOINED BY COMMON DIKES

connected, and multiple small independent SI's), the fill efficiency ratio concept was used to establish dimensions. Since optimizing construction cost would be one goal of the operator, the most cost effective dimensions for a given SI volume are those with the highest ratio of storage volume to fill volume. Therefore, the hypothetical SI's were modeled and optimized for highest fill efficiency ratio.

Survey work by Geraghty and Miller (1980) indicated that the majority SI's are sited near stream valleys, tidal estuaries, or bays. The study also indicated that most SI's are located in unconsolidated sediments over near-surface groundwater that discharges directly to surface water bodies. Consequently, failure of a liner system could cause rapid and extensive environmental contamination. Increased awareness of the water quality threat posed by SI's is reflected in the Hazardous and Solid Waste Amendments of 1984 (HWSA). A new disposal SI must now have a double liner system and groundwater monitoring while most existing SI's must be closed or upgraded to current liner standards within four years of enactment of HWSA. The systems chosen for the scenarios in this study are three with double liners and one with a triple liner (i.e., double flexible membrane liner - clay) (Figure 3).

COMPARATIVE CONSTRUCTION COSTS

Replacing one large SI with multiple small SI's has a major impact on the cost of SI construction. The optimum dimensions of the SI's in the scenarios evaluated in this study were used to estimate generic engineering and construction costs; unit cost data were taken from current estimates by the EPA (Craig, 1984). Table 1 lists the costs for double flexible membrane lined (FML) impoundments. Cost of four small SI's would be nearly double that of one large SI. The relative costs of each scenario holds for the other three liner scenarios shown in Figure 3.

SUBSURFACE FAILURES

All SI configurations examined in this study had either double or triple liner systems. Consequently, subsurface failures are considered due to multiple liner failures. While there are numerous types and brands of liners, only generic failure mechanisms for FML's and clay liners were considered. Both liner types can be made to leak excessively if they are incompatible with the waste, improperly designed, improperly installed or misused. Excessive lateral leakage can reduce the structural stability of earthen dikes (see following section on surface failures). Excessive vertical leakage is primarily a risk due to its potential for contaminating groundwater and downgradient surface water.

In the case of SI's lined with FML's, the smaller the SI, the greater will be the length of seams per unit SI volume. For the large SI used in this study, there would be a total seam length of 2050 meters. Each smaller SI would have a total seam length of

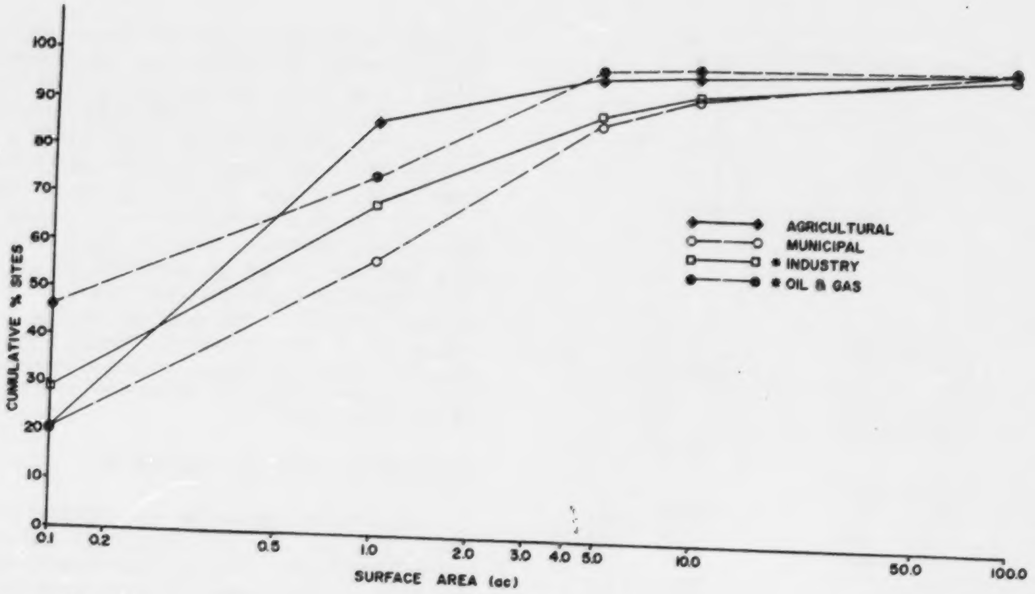

FIGURE 2. SURFACE IMPOUNDMENT SIZES

Table 1. Cost Estimate Parameters for Double FML Liner System

Parameters	Large SI	One Small SI	Four Small SI's
Total Excavation Volume (M^3)	12,286	4,365	17,460
Layer 1			
30 mil FML (M^2)	16,384	6,053	25,079
Geotextile (M^2)	11,621	3,969	15,876
Layer 2			
Sand (M^3)	4,839	1,771	7,078
Layer 3			
30 mil (M^2)	16,384	6,053	25,079
Layer 4			
Geotextile (M^2)	16,384	6,053	25,076
Sand (M^3)	2,420	886	3,539
Number of Leachate Branches	8	5	20
Leachate Laterals (M)	862	315	1,260
Leachate Main (M)	128	78	311

913 meters. Consequently, the four small SI's would require 178% of the seam length required for the one larger SI. Presuming leaks are more likely through seams due to inadequate quality control in the field seaming than through the middle of an FML sheet, there should be significantly fewer leaks per unit volume of waste with the larger SI. On the other hand, hydraulic head (total potential liquid depth) in the large and small SI's would be 320 cm and 230 cm respectively. Consequently, while there is a 39% greater hydraulic head in the large SI, there could be a 78% increase in number of leaks from the small SI's due to their greater seam length if leakage is proportional to total seam length. The result of these two factors should be a net 28%

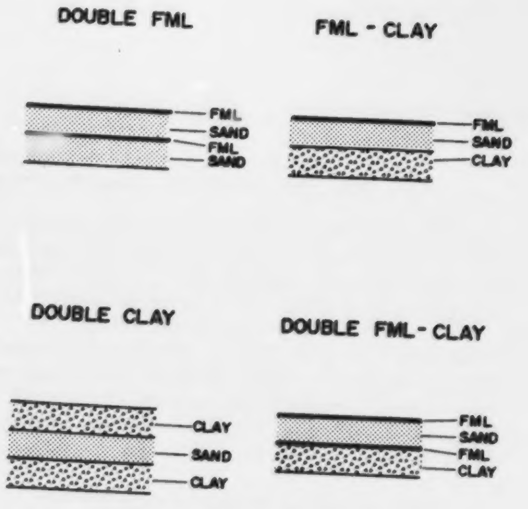

FIGURE 3. LINER SYSTEMS APPLICABLE TO SURFACE IMPOUNDMENTS.

greater leakage rate from the smaller SI's that are lined with FML's.

Assuming that both size SI's are lined with 60 cm of clay, the hydraulic gradients in the large and small SI would be 6.3 and 4.8 respectively. Considering only hydraulic gradient, the large SI should leak at a rate 31% greater per unit area than the small SI. The four small SI's, however, have a total bottom surface area 37% greater than that of the single large SI. Due to this additional bottom area, the multiple small clay lined SI's should have a total leakage volume of 5% more than the single large unit lined with clay.

SURFACE FAILURES

Performance of one large versus multiple small SI's was evaluated assuming occurrence of the various surface failure mechanisms, broadly categorized as either catastrophic dike failure or overtopping. An implicit dynamic wave flood routing model was used to compare resultant damages from the different failure types for the three SI design scenarios.

Catastrophic Dike Failures

Because of their dire environmental consequences, catastrophic dike failures are the major concerns for the performance of an SI. Such catastrophic failures can be caused by stability problems with the dike or by overtopping (discussed in greater detail below). Because little data exists to substantiate the overall performance and failure rate of SI's, this evaluation emphasized what is known about dike stability analysis and the factors influencing stability.

Stability of dikes is primarily a function of engineering properties of the material and quality control exercised in design and construction rather than cross-sectional area. Stability analysis is based upon soil cohesion, the angle of internal friction, pore pressures, density, and geometry of the dike material (Smith, 1982). When pore pressure is low, the remaining factors dictate the stability analysis (Whitlow, 1983). If double or triple liner systems are performing as designed, an insignificant amount of liquids should penetrate the liners and enter the dike materials. Therefore, when liners perform properly, increases in pore pressure due to liner leakage should be of little consequence to the stability of a given cross-sectional area of dike for small or large SI's. In the case of four small independent SI's there would be an apparent increased risk of dike failure as compared to the single large SI simply because of the additional external dike area.

When seepage from the liner system occurs, the stability of the dike will be reduced and may be threatened. With substantial leakage, material saturation and the resultant increase in pore pressure can cause a decrease in dike stability (Whitlow, 1983). Piezometers placed within the dike can be used to monitor fluctuations in pore pressure that may indicate a failure in the liner system. Liner leaks also present a problem with erosion of the dike beneath the point of leakage and can increase the risk of dike failure. In the case of dike failure, environmental damages tend to increase with the SI size. In any situation where risk of dike failure is heightened (e.g., earthquake zones or storage of waste that may damage a liner), there are advantages to using multiple smaller SI's.

In SI's with common dikes (Figure 1), the risk of failure is also increased by the rapid drawdown of liquid in an adjacent SI (Smith, 1982). A sudden change in loading of the adjoining dikes may place them in a less stable condition. The presence of double or triple liner systems should reduce this likelihood of multiple SI failure. However, if the first SI failure was due to liner-waste incompatibility and the other SI's have the same liners and waste, multiple SI failures could occur.

Overtopping

Overtopping of an SI can occur as a result of wind induced wave action or failure of level control systems. Comparison of these two causes for overtopping reveals that a level control device failure is a much greater hazard because of the rapidity with which it may result in catastrophic dike failure. Maximum liquid level should be controlled by an outfall mechanism that is relatively insensitive to inflow (e.g., a weir or a spillway). Freeboard (vertical distance from the top of the SI embankment to the maximum liquid surface) should be considered not as reserve storage volume, but as wave protection.

If losses of liquid due to wave action were considered alone, possible damages ensuing from such a discharge would be relatively minor. The resultant dike softening and wave cutting action may, however, cause catastrophic failure. To account for the risk of such major failures, current standards require that at least 2 feet of freeboard to be maintained (EPA, 1982). All models developed thus far for determining freeboard requirement are based on large water bodies with fetches (i.e., unobstructed diagonal lengths) of greater than 1 mile. Zabcik and Kissock (1983) reviewed these methods, and concluded that even the largest SI considered in this study would not be large enough to overtopping from wave action alone where two feet of freeboard is maintained.

As indicated previously, the use of some level control mechanism is necessary to maintain adequate freeboard in an SI. Since level control is a mechanical feature of SI's, the probability of failure of one of the four small SI's is four times greater than for one large SI. The degree of failure by overtopping is a function of the method employed for level control as follow:

1) automated or manually operated controls; and
2) automated or manually operated controls backed by a passive control such as a spillway.

Failure of controls in the former case would result in the loss of freeboard and in potential overtopping or levee softening that could lead to catastrophic dike failure. In such an event, the potential damage assessment would follow the same approach as for other catastrophic failure mechanisms (see the following section). If failure occurred in the latter case, the passive control system would allow a limited discharge of contaminants but would avert a catastrophic dike failure and massive waste release.

Resultant Damages

Failures can take two forms, catastrophic loss of all the above grade contents of the SI or limited losses from overtopping. Conditions leading to catastrophic failure include slope failure of dikes, overtopping and erosive cutting of dikes caused by level control failure, and rapid drawdown slope failure of one small impoundment in response to failure of another SI that shares a common dike (Figure 1).

An implicit dynamic wave flood routing model was developed to map potential inundation areas and to evaluate the relative effects of large versus small SI failures. This model transforms the partial differential equations of unsteady flow (Saint-Venant, 1871) into a set of nonlinear algebraic equations which are solved simultaneously by the Newton-Raphson iterative scheme. As a result of a breach in a dike, a discharge hydrograph is produced and used as the upstream boundary condition for solution of the model equations. The failure of dikes is usually neither complete nor instantaneous, with breach formation occurring in a matter of minutes to a few hours and with an average width of 1 to 3 times the dam height. Breach shape is modeled as rectangular, triangular, or trapezoidal. The downstream boundary condition is determined according to a rating curve of flow rates with corresponding depths of flow at downstream nodes. Initial conditions consisted of the base flow at the downstream nodes. Downstream geometry must also be specified to allow a solution of the unsteady flow equations.

Parametric studies were run with the model to determine the influence of the various parameters on waste flow. It was found that because breaches occur relatively quickly in dikes, the resultant peak flows occur within 0.5 to 3 hours. Rapid failure is significant in that, regardless of SI size, failures are likely to occur before the operator can take mitigative actions. Peak flows are roughly proportional to impounded volume and peak discharge increases with decreasing time of breach formation (Figure 4). The model is, therefore, useful for predicting peak flows to be used in the design of secondary containment structures to withstand the impact of the released waste materials. The model presents a strong case for the need for secondary containment structures around SI's which would be analogous to containment berms around tanks, since the effects of a failure of even a small SI would be felt miles downstream. Although a greater spread of materials would undoubtedly occur in the event of a large SI failure, any failure without secondary containment would result in widespread environmental damages.

The influence of impoundment size on damages from overtopping is a function of whether or not a spillway is a part of the design. Where no spillway exists, overtopping can lead to the kind of catastrophic

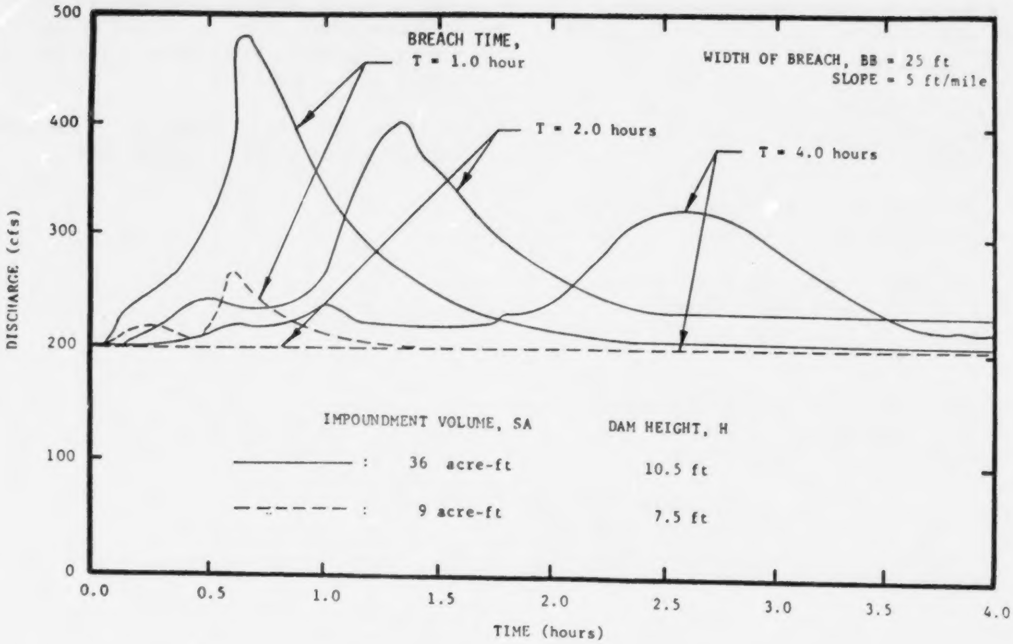

FIGURE 4. VARIATION OF DISCHARGE FROM BREACH WITH RATE OF BREACH FORMATION.

failure discussed in the previous sections. For designs that include spillways, impoundment size has an effect only if the overflow is from precipitation. In this event, the volume of material released is a function of SI surface area since area controls the volume of precipitation caught within the SI. If the only source of the flow is incoming waste or wastewater, volume lost over the spillway is independent of impoundment size.

CONCLUSIONS

The relative risks of one large versus multiple small SI's are summarized as follows:

1) For functional liner systems, there is no significant difference in failure probability of a given unit length of dike since, in such cases, failure is primarily due to engineering properties of the materials, design and construction and not dike size. There may be a greater overall risk of failure in the multiple SI case due to the greater external dike length, however, the extent of resultant damage from either SI size would be widespread;
2) If major liner leakage does occur, the depth of liquid impounded increases the probability of dike failure. Again, however, the resultant damage from either case would be substantial;
3) Only when overtopping caused by precipitation occurs with a spillway present does larger SI size increase the quantity of material lost; however, the risk of failure is greater for multiple SI's since a greater number of level control mechanisms are in place.
4) Subsurface leakage would probably be greater from multiple small SI's than from a single large SI due to differences in total bottom area and, in the case of FML's, seam length (per unit area) if each is constructed with the highest fill efficiency ratio. This is because optimal SI depth (and hence hydraulic gradient) increases with SI size.

With the following design features included, the risks of large or small SI can be greatly reduced.

1) spillways for controlling damages from uncontrolled releases;

2) piezometers to monitor pore pressures in the dikes;

3) secondary containment structures to limit contaminant spread in the event of SI failure;

4) for multiple small SI's separate independent structures rather than common dikes.

Where these features are included in the SI design, no real advantage is gained from the use of multiple impoundments rather than one large structure.

ACKNOWLEDGMENTS

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UPDATE: STORAGE OF HAZARDOUS WASTE IN MINED SPACE

(Results of work conducted under U.S. Environmental Protection Agency Contract #68-03-3191)

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ABSTRACT

This report presents the results of an assessment of the current status of using mined space for long-term retention of non-radioactive hazardous waste. The report is an update of previous studies conducted from 1974-1977, and examines: recent literature published on the subject, recent project activities, involvement of government agencies, review of regulatory and permitting requirements, selection of existing mines for a demonstration project and presents a new approach proposed by industry which is the use of solution-mined space for retention of hazardous waste in salt.

INTRODUCTION

This paper presents highlights of the study conducted under U.S. Environmental Protection Agency Contract #68-03-3191 entitled, "Using Mined Space for Long-Term Retention of Non-radioactive Hazardous Waste."

With enactment of the 1976 Resource Conservation and Recovery Act (RCRA), and the hazardous waste regulations promulgated by the EPA, a national hazardous waste management program exists.

The presently accepted methods of disposal of hazardous waste are deep well disposal, engineered landfills, land treatment of hazardous waste and incineration.

After all present treatment and storage methods have been tried and found to be lacking in one aspect or another, there remains the need for an ultimate disposal technique or method which can handle the residues from the other methods.

Use of selected, existing mined space appears to be, technically and economically, a feasible method of permanent storage of untreatable wastes or the toxic end products of hazardous waste treatment.

Among the advantages of mined space for hazardous waste storage are:

- In deep mined space the waste would be below drinking water aquifers.
- Isolation from the public and the surface ecology.
- If required, waste can be isolated from hydrological environment by encapsulation or containerization.
- Security can be readily maintained.
- In a sealed mine, no continuing maintenance will be required.
- If retrievability is desired, the mine could be used as a long-term underground warehouse.

PURPOSE

The purpose of this project was to update the state of the art of using mined space for hazardous waste storage in the U.S. A secondary purpose was to determine the feasibility of a limited demonstration project.

APPROACH

The approach included review of activities conducted during the past ten years to describe changes in the state-of-the-art related to the use of mined space for retention of hazardous waste. This review was based on a literature search to identify past and present research, government activities, activities associated with the development of commercial facilities, and foreign activities involved with the use of underground space for storing/disposing of wastes.

The approach also included a review of federal and state regulatory requirements which would be involved in obtaining a permit for an underground hazardous waste disposal facility.

The selection of candidate mines was based on a review of past government and private client studies and reports. This assessment indicated a number of conventional mined spaces in limestone, salt, potash and gypsum mines which might have potential for hazardous waste storage.

The literature search indicated that solution-mined storage caverns in salt offer additional storage possibilities.

Literature Search: U.S. and Foreign

The first step in reviewing the literature as to the status of non-radioactive hazardous waste storage in mined space was a data base literature search. Publication citations were retrieved for literature published since 1974.

Recent Activities: Foreign

Germany: The Herfa-Neurode facility continues to operate satisfactorily 12 years after its start-up in 1972. Approximately 270,000 tons of hazardous waste were placed in the mine in ten years, and the present annual volume is running between 35,000 and 40,000 tons. Approximately 25% of this tonnage originates outside of Germany. The reuse of stored waste is possible, and over 1,000 tons has been retrieved by a waste producer and returned to the originator for further use.

India: Studies for storage of hazardous waste in mined space have been conducted for the central region of India as reported by K.D. Ghosh of the Geological Survey of India, in Rockstore, 1980.

Recent Commercial Activities - United States

Louisiana: Empak, Inc. of Houston, Texas applied to the Louisiana Department of Conservation for a permit to build a hazardous waste facility on the Vinton Salt Dome in southwest Louisiana. This project envisioned using solution-mined caverns in the salt dome as final storage for hazardous waste. After this project was announced, a state law was passed (1983) forbidding emplacement of hazardous waste in salt domes for a period of two years. This was intended to allow the state time to evaluate the proposed use prior to issuing a permit.

Texas: United Resource Recovery, Inc. of Houston submitted an application to the Texas Department of Water Resources for a permit to store hazardous waste in the Boling Salt Dome. The application was returned to the company for further elaboration and information, has subsequently been resubmitted and was pending (January 1985). This request has, in turn, triggered a geological study to evaluate acceptability of using salt domes in Texas for waste disposal, and recommended guidelines for waste storage in domes. This work is being done by the Texas Bureau of Economic Geology and will continue until August 1985.

Minnesota: The Minnesota Waste Management Board financed a study entitled, "Subsurface Isolation of Hazardous Wastes" which was completed in 1982. This study was conducted by the University of Minnesota and was oriented to deep geologic disposal in crystalline bedrock within the state. Following preparation of the report, the Waste Management Board made a survey to locate specific sites. They identified 18 mine study sites and anticipated reducing these to five sites for further investigation. The Minnesota Waste Management Board dropped the Crystalline Rock Concept from further consideration due to its high cost during a meeting held on February 24, 1984.

Ohio: Proposed conversion of the PPG Industries, Inc.'s (Barberton) limestone mine to a waste storage facility in 1981/82 has been dropped due to U.S. economic conditions. A public notice to this effect was issued by PPG Industries on July 12, 1982.

Research Activities

The U.S. Environmental Protection Agency has sponsored several studies and development projects in recent years covering the fields of encapsulation, containerization and fixation technologies. The developments from these projects have provided valuable background information on containment which did not previously exist and which would apply directly to the use of mined space for retention of hazardous waste.

The American Society for Testing Materials (ASTM), through its Committee D-34, has been working on a "Guide for Determining the Compatibility of Hazardous Waste." Once acceptable methodology has been finalized the determination of waste to salt environment compatibility must be established for mined space storage.

Public Reaction to Hazardous Waste Facilities

The press and media have produced continuing commentary of activities associated with generation, transportation and storage of hazardous wastes. The public has consequently become increasingly aware of the potential for damage to all life forms from these wastes. This public awareness apparently does not presently include a general recognition of the national consequences if some type of final treatment or storage is not developed.

INVOLVEMENT OF GOVERNMENT AGENCIES

Every phase of a hazardous waste storage facility, from conception to closure, will require involvement of government agencies at all levels: federal, state and local. These agencies and the extent of their involvement will vary with location and time as new legislation is passed and the programs are implemented. Major issues concerning government involvement with a hazardous waste storage facility are the protection of the public and the process of siting new facilities.

Regulation and Permitting Assessment

In May 1980, EPA published regulations in accordance with Subtitle C of the 1976 RCRA, as amended. These regulations include provisions whereby a state agency may be authorized by EPA to administer a hazardous waste program in that state in lieu of a federally-administered program. Many of the states have chosen to develop and manage their own hazardous waste programs under EPA regulatory guidelines. A few states have not developed or are lagging in development of their hazardous waste management plans, and EPA is now administering their programs. The result is that a reasonably uniform hazardous waste management program is being conducted throughout the United States.

At this time neither EPA nor any of the states have developed regulations specifically pertaining to hazardous waste storage in mined space. Therefore, initial mined hazardous waste facilities will have to be designed, sited and evaluated by interpretation of the intent in existing regulations, with ultimate legislation being developed if feasibility of mined storage is demonstrated.

Current Applicable RCRA Permitting Requirements

The EPA regulation governing permitting of new hazardous waste facilities is present in 40 CFR §270. This regulation requires preparation and submittal of a two-part application for approval of the proposed facilities.

Present permitting requirements do not include specific terms related to waste storage in mined space. The general format and requirements are, however, comprehensive enough to provide the basis for requisite information for a mined hazardous waste storage facility permit application.

State Hazardous Waste Permitting Requirements

For those states authorized by EPA to administer a hazardous waste program, permitting regulations have been developed and are presently being employed. For states without such EPA authorization, EPA regulations are utilized.

In general, all authorized states now have regulations and permitting practices which conform to EPA hazardous waste management regulations and administrative criteria. Differences in storage regulations and permitting procedures do not appear to be major. At this date, none of the authorized states have regulations which specifically relate to the storage of hazardous waste in subsurface mined space.

The 1976 RCRA does not prescribe requirements for the siting of new hazardous waste management facilities. Present policies consider this the responsibility of each state.

In the past, several state legislatures gave authority to various agencies for permitting new hazardous waste facilities proposed by industry. After analyzing the applications and conducting public hearings, the state would rule on the permit. However, public opposition to new facilities resulted in very few approved facility permits. In response to these problems, a majority of the states have developed siting programs which specify the process to be followed in selecting a hazardous waste storage site, provide for public participation and establish the responsibility of state and local agencies.

MINE SELECTION

The concept of hazardous waste storage in mined openings is not new but, other than one known instance in Germany, has not been utilized. An EPA study conducted by Fenix & Scisson, Inc. in 1974 concluded that the concept was technically and environmentally acceptable, provided, however, that recommended site selection and waste management procedures were followed.

Because mined space is expensive to construct and a demonstration of the technique is needed, a viable alternate to construction of new mined space is to utilize existing mined space in older mines.

The purpose of this task was to select three candidate mines with good potential for use as a demonstration facility for retention of non-radioactive hazardous waste. The basic criteria specified by EPA were:

- The preferred candidates were to be salt mines; however, other mines were not ruled out.
- The mine should have been mined by the room and pillar method.
- Existing engineering, geology and other information indicates it is suitable for the purpose.
- An industrial organization willing to cooperate in a limited demonstration of the concept must exist.

Additional criteria developed included:

- Hazardous wastes to be stored must be packaged, encapsulated or solidified, non-reactive, dry, noncombustible, and untreatable by current technology or the end product of current treatment technology.
- Fenix & Scisson elected to include limestone mines, mined by the room and pillar method, in this report.

The approach utilized in selecting three candidate mines for a demonstration project consisted of:

Identification

A status list of all known salt mines in the United States was compiled from various records and sources. In addition, four limestone mines were listed which have recently been studied as possible storage sites for the Department of Energy's Petroleum Reserve Program. This listing was subdivided into operating and shutdown mines.

Analysis

The selection of candidate mines required consideration of a multiplicity of factors. The method used in the selection process consisted of:

- A preliminary screening of 30 potential mines for storage use to eliminate obviously unsuitable candidates (such as flooded, used for other purposes, or inaccessible).
- Geographical: The mines were located geographically and their location matched to the EPA Regions where hazardous waste is generated.

Geographical Screening

Figure 1 shows the geographic location of the remaining 15 mines and the EPA Hazardous Waste Reporting Regions. Two of the highest volume regions as reported by EPA are Region V and VI. These regions will provide adequate volumes of "mine acceptable" hazardous waste for a demonstration project and are located over major salt deposits. The character of wastes generated in Region V is anticipated to be predominantly manufacturing waste from the concentrated automobile industry in the Cleveland, Toledo and Detroit areas. The character of the wastes generated in Region VI is anticipated to be predominantly petrochemical wastes from the concentration of oil related industry in this region.

Rating Remaining Mines

The third step in identifying candidate demonstration mines was a rating of the surviving seven mines located within EPA Regions V and VI. The rating method used was subjective and included the following considerations:

- Geological
- Hydrological
- Marketing Factors
- Socio-political
- Accessibility
- Location
- Previous Studies
- Owner Interest

Additional Investigation

The three salt mines selected as

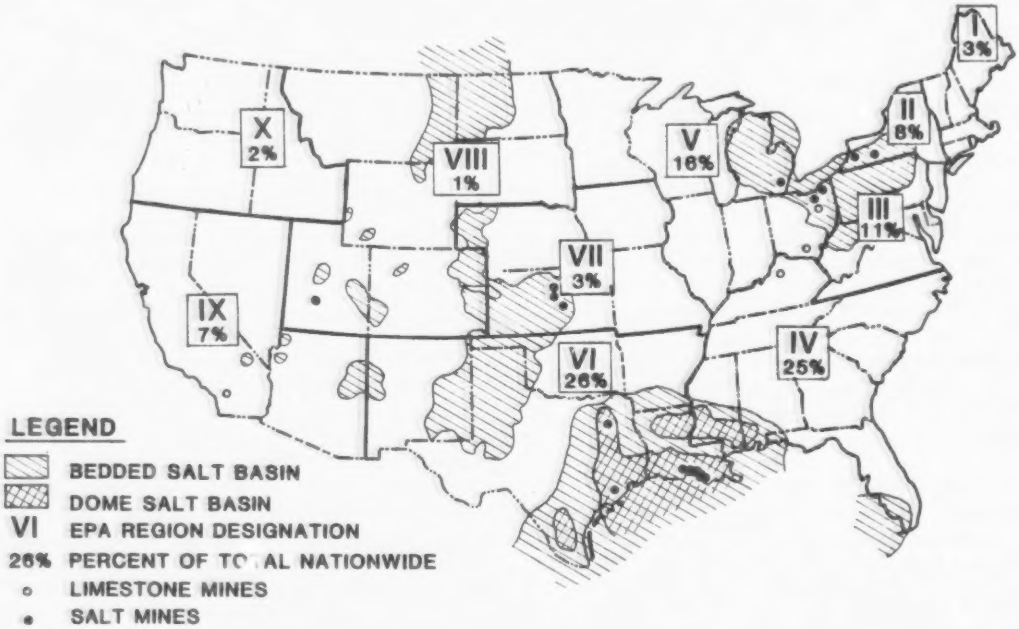

Figure 1. 1980 Industrial Hazardous Waste Generation Within Each EPA Region. (Percent of Total Nationwide)

candidates are located in Michigan, Ohio and Texas. The alternate (limestone) mine is in Ohio.

The next step of the selection process was to visit each facility and discuss the proposed scope of the demonstration with the respective owners/operators of the facility.

One mine was withdrawn from further consideration when the operators indicated "it was not in their best interest" to participate in the project.

The second mine studied was the subject of adverse public reaction to a recent attempt to establish a commercial hazardous waste storage facility in the mine. It was concluded that this would be detrimental to any effort for establishing a demonstration facility. Trying to establish a demonstration project immediately after the previous effort would only stiffen public resistance and make the probability of obtaining a state permit for the facility all the more difficult.

The third mine was the highest ranked of the original three candidate mines, and the owner has indicated willingness to participate in the demonstration. The next step in the process was to determine what problems would be encountered from state permitting agencies in establishing a hazardous waste demonstration project. It was found that the state has jurisdiction for hazardous waste facility siting and that their regulations are equal to, or more stringent than, those required by the 1976 RCRA. There is no provision in any of the existing state or federal laws or regulations for research and development efforts, such as a demonstration facility, and that the proposed facility would have to go through the same permitting procedures under the existing state regulations as any other hazardous waste facility.

An alternate limestone mine was ranked numerically at the top of the list and illustrates the possibility of utilizing mines in other types of rock rather than salt. This particular mine has been shut down for many years and would require a hoisting system to be installed prior to any use or inspection. As a hoisting system is expensive to acquire and install, and due to the salt mine having an equal rating and an operating hoisting system for access, this mine was not considered further.

The third mine discussed is the best candidate for a demonstration Hazardous Waste Storage Facility.

SOLUTION-MINED STORAGE

Solution-mined caverns could provide secure containment for hazardous wastes and complete isolation of those wastes from the environment. After the waste slurry cavern has been plugged with cement, the plastic nature of the salt under strata pressure will seal the wastes more or less permanently. A sealed cavern would not entail future maintenance or security costs.

Large salt deposits are located near a number of the major hazardous waste generation areas, minimizing transportation and handling costs. In most EPA regions it should be possible to provide solution-mined retention for all of the liquid or slurry wastes generated within or near that region. Solution caverns have a long history of effective containment of hazardous products.

This may be one of the most economical methods of providing permanent retention of hazardous waste due to the fact that encapsulation would not be required.

Hydrocarbon Storage

No discussion of solution-mined salt caverns would be complete without mention of the vast quantities of hydrocarbons stored in them. Although these hydrocarbons are not hazardous wastes, absolute containment has been required and proven. In 1983 there was a total of over 520 million barrels (42 gallons/barrel) of storage capacity in the United States for propane and other light hydrocarbons. In addition, there are 400 million barrels of crude oil presently in storage in the U.S. Strategic Petroleum Reserve. With nearly one billion barrels of solution-mined storage capacity available in the United States, it should be apparent that industry has the utmost confidence in the containment and security of this concept.

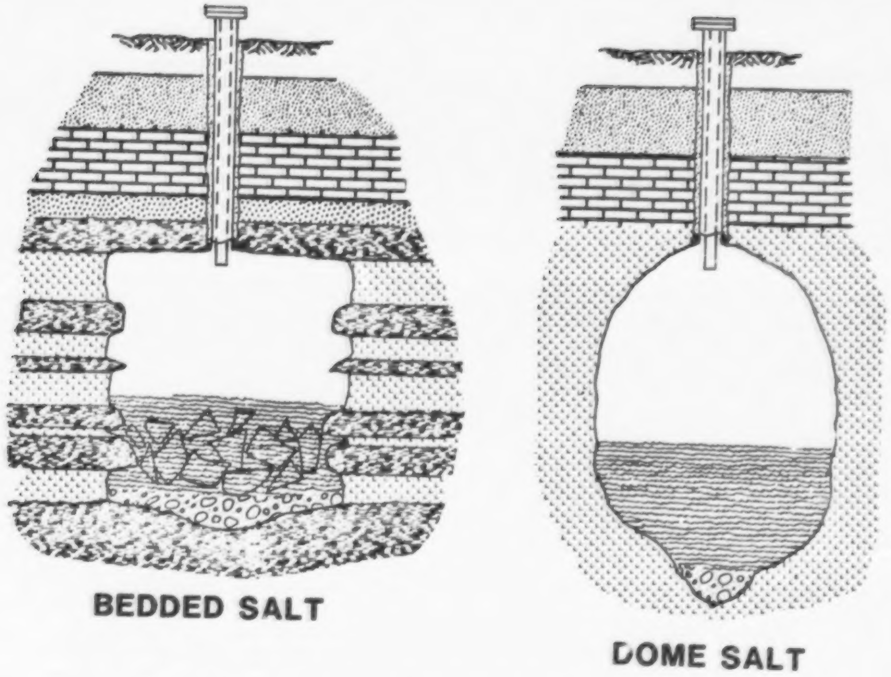

Figure 2. Solution Mined Caverns In Salt.

Basic Requirements for Solution-Mined Storage Caverns

Three basic conditions must exist at the location where storage facilities are needed if solution mining is to be used for cavern construction:

1. A sufficient thickness of structurally competent salt at a proper depth, without an excess of interbedded insolubles, must be present.
2. An adequate supply of raw water for leaching the salt must be available.
3. An environmentally acceptable and economical means of disposing of the resultant brine must be available.

Salt Dome Leaching Techniques

The fundamental technique of salt dome cavern development is to drill into a salt mass and to pump "raw" (fresh or low salinity) water into the hole, thus dissolving the salt which is then carried to the surface as a brine solution. The hole within the exposed salt gradually enlarges, eventually forming a useful cavern. In actual practice, the procedure for cavern development is somewhat more complex.

Bedded Salt Leaching Techniques

The basic principles of leaching such as the use of blanket material, direct and indirect circulation, etc., are the same in bedded salt as in a salt dome. Bedded salt, however, presents additional problems due to lack of thickness of the salt resource, and also due to the tendency of large interbedded shale layers to collapse and damage the leaching casing strings. In general, the bedded salt insolubles tend to fall to the bottom of the cavern as a pile of large rocks, rather than as fine sand as is the case in a dome caverns. Figure 2.

WASTE GENERATION BY EPA REGION VS. SALT DEPOSITS

Of the ten EPA regions, the four largest producers of hazardous waste are III, IV, V and VI. In 1980 these four regions produced 78% of the nationwide total.

Regions IV and VI, with 25% and 26% of the nations hazardous waste, respectively, both have salt domes which could be utilized for the safe retention of hazardous waste.

Regions III and V, with 11% and 16% of the nations hazardous waste, respectively, both have bedded salt which could be utilized. Although the bedded salt caverns would be smaller and more numerous for the same volume of storage in comparison with dome salt caverns, shipping costs would be reduced and they might be cost effective on an installed cost per barrel basis, if a method of brine disposal can be developed.

METHODS FOR STORING HAZARDOUS WASTE IN SOLUTION MINED SALT CAVERNS

There are a number of methods which might be employed for utilizing solution mined caverns for the storage of hazardous wastes.

Brine-Balanced Cavern with Brine Discharge During Waste Injection

Nearly all of the existing solution mined storage for liquid hydrocarbons in the U.S. is brine-balanced. The propane or crude oil, being lighter than brine, floats on top of it and flows under pressure when the product valve at the wellhead is opened. During product withdrawal, brine is added to the casing string hanging to the bottom of the cavern so that the cavern is maintained full of fluid at all times. When product is introduced into storage, it must be injected by means of a pump, displacing brine up the casing string and over to a brine holding pond. Caverns using this concept have a very long life because the range of stress on the salt is small.

The principal advantage if this method could be used for hazardous waste storage would be the possibility of utilizing very large caverns.

If the hazardous waste slurry were significantly lighter than saturated brine, the waste could be injected into the top of the cavern, displacing saturated brine up the casing string as shown in Figure 3. The displaced brine could then be injected into the same disposal wells that were used for construction of the cavern.

If the specific gravity of the waste were close to or greater than that of saturated brine, the waste slurry would have to be weighted so that it would be significantly heavier than the brine causing it to remain in the bottom of the other cavern as shown. The floating brine would be displaced from the top of this cavern and would be directed to disposal as before.

Although gravity segregation of liquids is used extensively in solution-mined caverns, the risk of contaminating the brine would increase as the specific gravity of the waste approaches that of the saturated brine. Brine contamination might also result from gases or vapors released by the hazardous waste in the cavern in the case of the heavy waste/floating brine cavern.

Gas-Balanced Cavern with Zero Discharge

In this method, after the cavern had been washed to capacity, the brine would be displaced by an inert gas and injected into a remote brine disposal well as shown in the cavern dewatering scheme of Figure 4. The cavern would be sealed at the minimum design pressure after which gaseous, liquid and slurry wastes would be injected into the cavern until the inert gas reached the maximum design pressure of the cavern as shown in the storage scheme of the same figure.

Gas-balancing might allow the use of a larger cavern than the atmospheric method, but smaller than the brine-balanced cavern. Gas-balancing also eliminates the possibility of contaminating the brine, and eliminates the need for a scrubber and flare, unless they were needed by the surface plant.

Atmospheric Cavern with Controlled Gas Discharge During Waste Injection

Following solutioning of the cavern

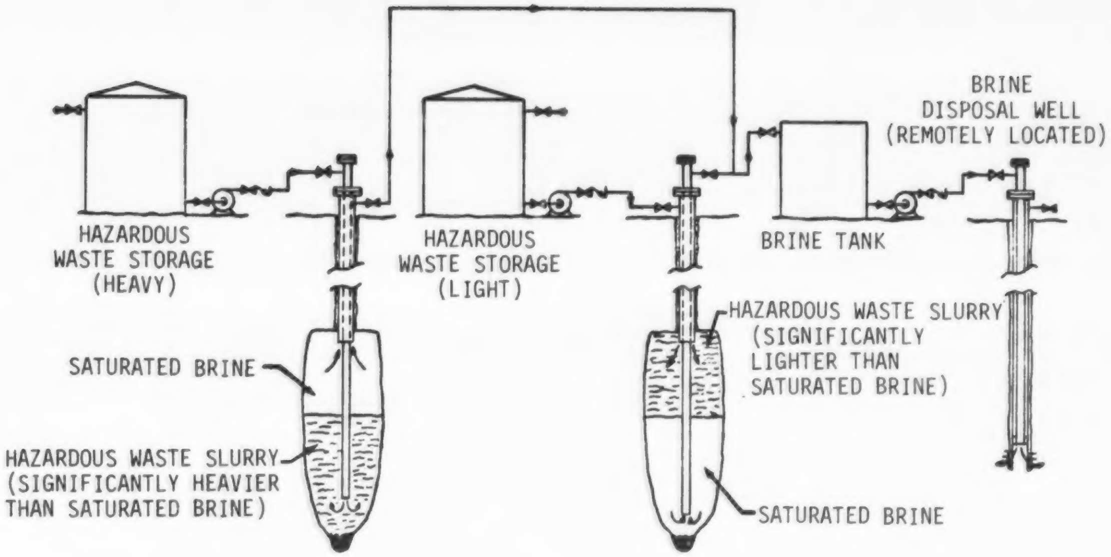
BRINE
DISPOSAL WELL
(REMOTELY LOCATED)
HAZARDOUS
WASTE STORAGE
(HEAVY)
HAZARDOUS
WASTE STORAGE
(LIGHT)
BRINE TANK
SATURATED BRINE
HAZARDOUS WASTE SLURRY
(SIGNIFICANTLY
LIGHTER THAN
SATURATED BRINE)
HAZARDOUS WASTE SLURRY
(SIGNIFICANTLY HEAVIER
THAN SATURATED BRINE)
SATURATED BRINE

Figure 3. Brine-Balanced Cavern Cavern With Brine Discharge During Waste Injection.

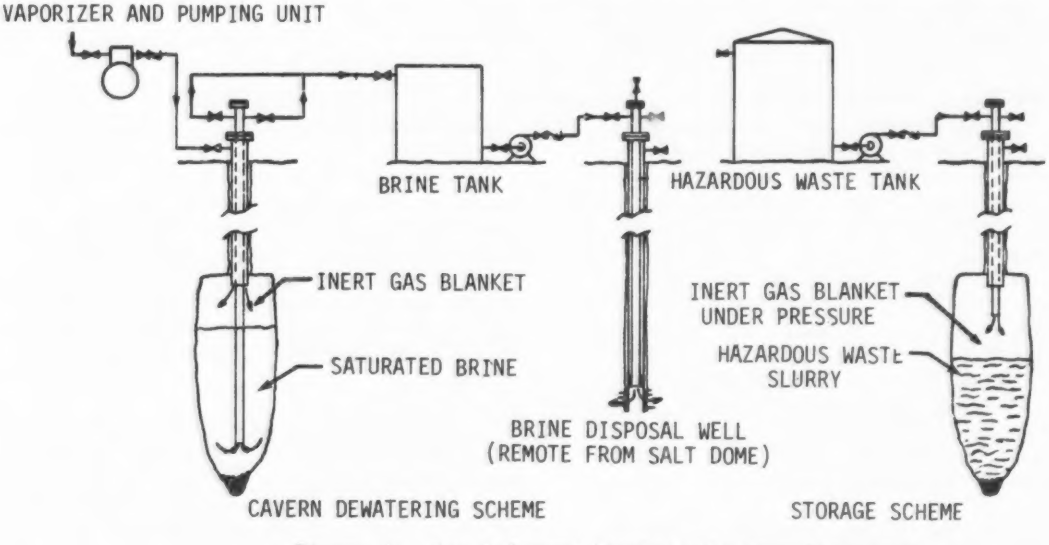
VAPORIZER AND PUMPING UNIT
BRINE TANK
HAZARDOUS WASTE TANK
INERT GAS BLANKET
SATURATED BRINE
BRINE DISPOSAL WELL
(REMOTE FROM SALT DOME)
INERT GAS BLANKET
UNDER PRESSURE
HAZARDOUS WASTE
SLURRY
CAVERN DEWATERING SCHEME
STORAGE SCHEME

Figure 4. Gas-Balanced Cavern With Zero Discharge.

to its design capacity, the brine could be pumped out of the cavern by means of a submersible pump and directed to disposal. Chemically compatible liquid and pumpable slurry wastes would then be inserted into the cavern. Displaced vapors and/or gases would be collected and would be either sent through a scrubber or burned in an approved flare as shown in Figure 5.

A cavern designed to be exposed to atmospheric pressure would be limited in size to maintain structural integrity. There would be no brine contamination, but a scrubber and flare would be required.

In-Situ-Solidified Waste Storage

Recently, officials of the state of

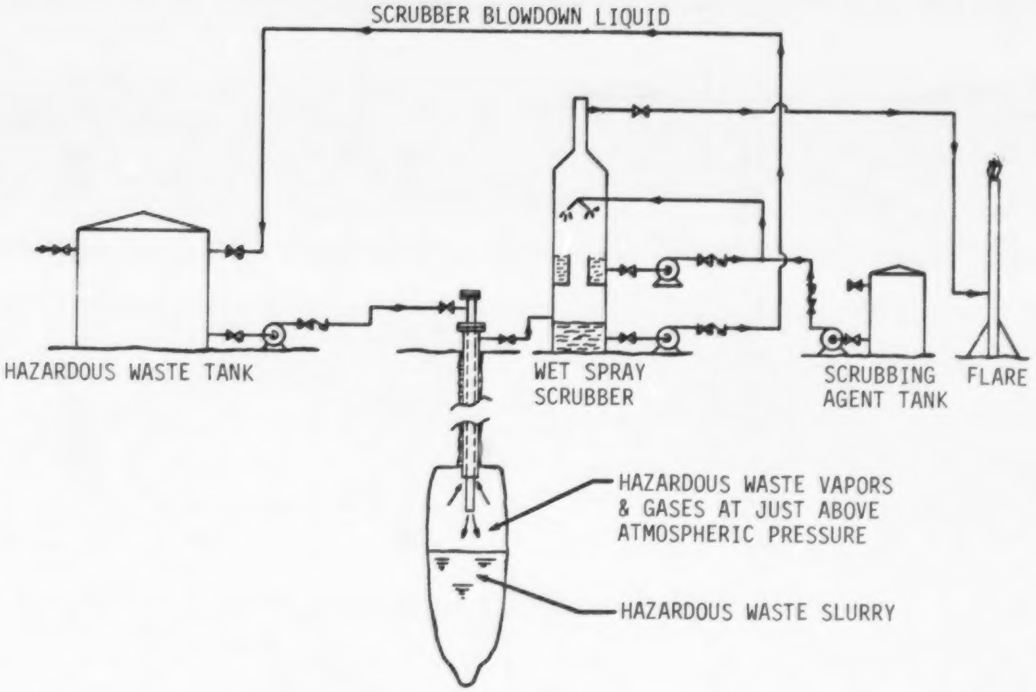

Figure 5. Atmospheric Cavern With Controlled Gas Discharge During Waste Injection.

Texas, who are responsible for permitting requirements for underground storage facilities, have been considering a concept that would require that all hazardous waste be mixed with a cement or polymer slurry prior to injection in a cavern as shown in Figure 6. This would provide a permanent solidified storage for liquid and slurry wastes and would reduce risks occasioned by earthquake or the inadvertent drilling into a hazardous waste cavern. This cavern would be limited in size in order to maintain structural integrity during the period that it would be exposed to atmospheric pressure.

"String-of-Pearls" Waste Storage Caverns

By constructing a series of caverns, one above the other, from one deep solution well, the dome salt resource could be more effectively utilized for a given maximum cavern depth as proposed by Empak, Inc. of Houston, Texas. Each cavern could be sealed by the installation of a cement plug in the top neck of the cavern, prior to starting construction of the next cavern above it as shown in Figure 7.

The brine from the first (bottom) cavern could be removed by submersible pump and directed to the remote brine disposal wells. This cavern would remain structurally stable due to its small size and because it would be filled with waste relatively quickly, reducing the time that it would be exposed to near atmospheric pressure.

For maximum structural integrity into the future, all of the hazardous waste could be mixed with a cement slurry so that the entire small cavern would solidify as outlined by the proceeding method. Then construction of the small middle cavern could proceed as shown.

ENVIRONMENTAL, SOCIOLOGICAL, AND ECONOMIC CONSIDERATIONS

A hazardous waste retention facility must protect the environment at all times; during construction of the facility, during the process of adding waste to

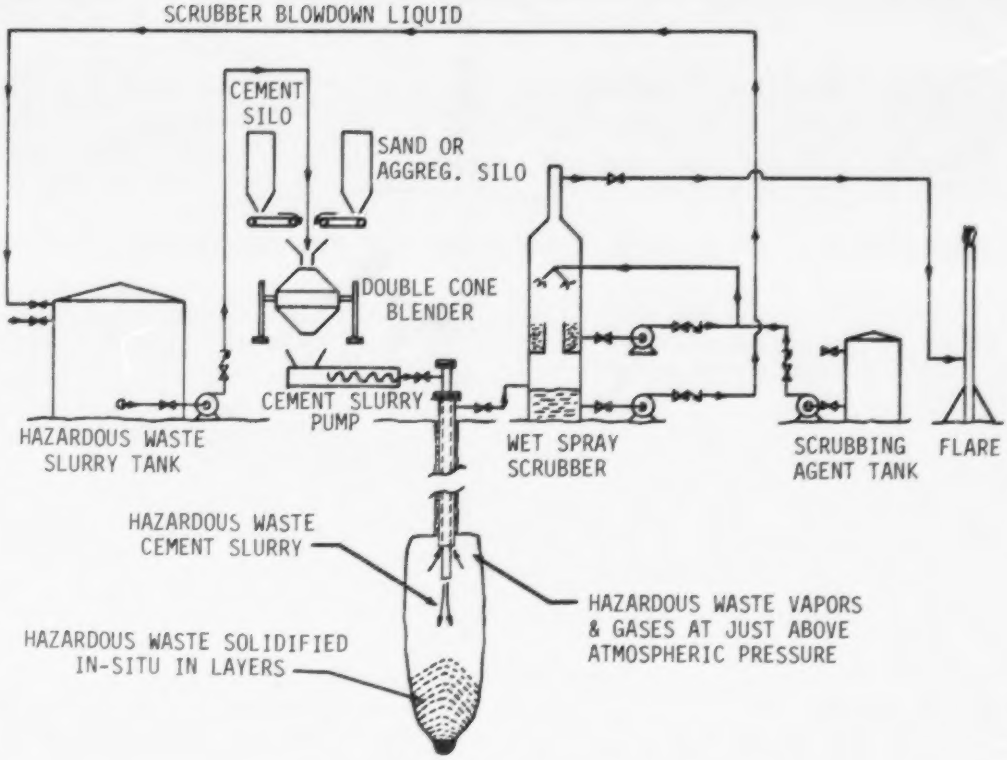
SCRUBBER BLOWDOWN LIQUID
CEMENT SILO
SAND OR AGGREG. SILO
DOUBLE CONE BLENDER
CEMENT SLURRY PUMP
HAZARDOUS WASTE SLURRY TANK
WET SPRAY SCRUBBER
SCRUBBING AGENT TANK
FLARE
HAZARDOUS WASTE CEMENT SLURRY
HAZARDOUS WASTE VAPORS & GASES AT JUST ABOVE ATMOSPHERIC PRESSURE
HAZARDOUS WASTE SOLIDIFIED IN-SITU IN LAYERS

Figure 6. In-Situ-Solidified Hazardous Waste Storage

storage, and for an indefinite period of time into the future.

All aspects of the environment must be protected; the land, water, air, noise level, etc. Solution mining offers a unique solution to this requirement.

A minimum of land surface would be required for the concept. Many salt domes are relatively free of population and industry. The exceptions are industries utilizing the salt such as salt producers or hydrocarbon storage companies and on the flanks of some salt domes, drilling rigs.

Construction of the facility would have a minimal impact on the local area. Construction at the site would increase traffic and background noise slightly due to drilling, building, and tank erection activities and machinery operation.

Conservative design would prevent the possibility of subsidence.

Operation of the site should not have an adverse effect on water quality since there will be no discharge to surface waters.

The clean, saturated brine produced as a result of solutioning storage space could be injected into deep disposal wells.

The facility could be manned 24 hours per day, 7 days per week to provide constant monitoring.

Sources of air emissions from the facility would be limited to the diesel blanket storage tank.

Vapors displaced from the caverns would be chemically scrubbed or burned in

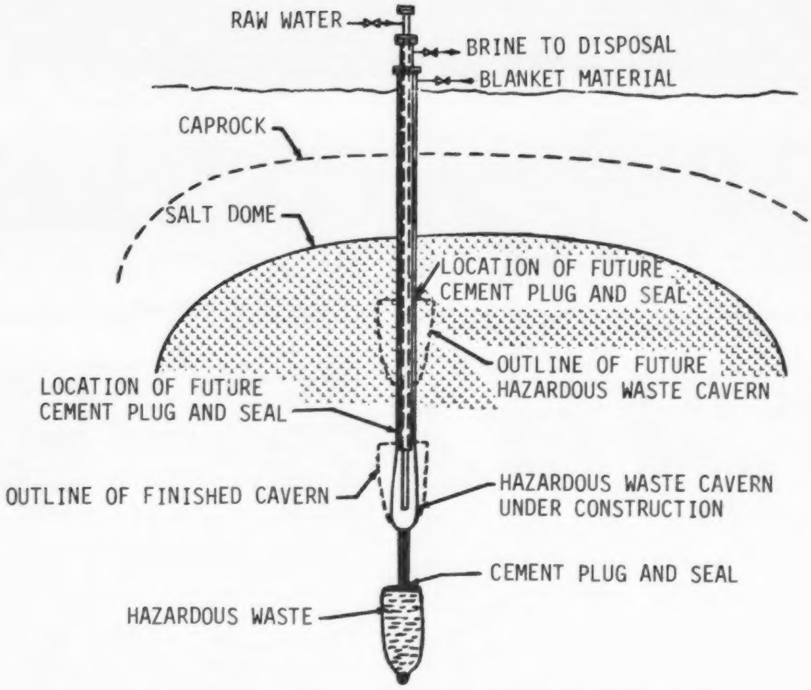
RAW WATER
BRINE TO DISPOSAL
BLANKET MATERIAL
CAPROCK
SALT DOME
LOCATION OF FUTURE CEMENT PLUG AND SEAL
OUTLINE OF FUTURE HAZARDOUS WASTE CAVERN
LOCATION OF FUTURE CEMENT PLUG AND SEAL
OUTLINE OF FINISHED CAVERN
HAZARDOUS WASTE CAVERN UNDER CONSTRUCTION
CEMENT PLUG AND SEAL
HAZARDOUS WASTE

Figure 7. "String-of-Pearls" Waste Storage Caverns.

an approved flare.

Careful siting of the facility would ensure no wildlife habitat would be disrupted in the immediate areas of construction of tanks, operational areas, and pipelines. Major facilities would be located in areas that are already relatively clear of vegetation. The storage facility would not be located in any wetlands area.

NEEDED RESEARCH

Solution mined salt caverns have been used for storage and disposal of certain hazardous wastes by industry for many years. The practice has been limited to compatible chemicals which are known to be nonreactive to each other and to the salt. One problem faced by the hazardous waste industry is the almost infinite mix of chemicals which occur in many waste discharges. Efforts have been made by the American Society of Testing Materials (ASTM) to determine compatibility of the various chemicals based upon theoretical analyses. This approach is helpful but all possible combinations may not have been adequately described and unexpected reactions may occur. When a commercial hazardous waste facility has many clients, each with a different waste stream, the potential for adverse reactions is multiplied many times if these waste streams are mixed together for a disposal operation. Thus, more research including both studies and actual tests of compatibility should be conducted for each combination of waste streams. When placed in the retention cavern, the waste will be in direct contact with the salt and with saturated brine. Research is needed to determine the analysis of salt cores taken from proposed solution cavern areas and to determine the compatibility of each with a mix of anticipated waste streams. If salt cores are not available from a given area, blocks of salt from a mine in the area could be substituted. Samples of typical waste streams could be obtained from commercial disposal operators or waste generators. These tests could be run in a laboratory under controlled conditions. Tests should be run on salts from the Permian basin and Salina basin, and at least two salt domes.

Solution mined salt caverns would be even more desirable for hazardous waste retention if a means could be developed to solidify the waste in-situ in the saturated brine storage cavern. Small scale research studies and laboratory tests are needed to develop salt saturated hazardous waste/cement or polymer slurry formulations that will have an acceptable fluidity during emplacement in the cavern, but that will develop adequate strength and long life with acceptable economy when solidified in the retention cavern. After the waste has been emplaced in the cavern, there must also be a practical way to flush the surface piping and well tubing before solidification occurs.

Preliminary feasibility studies including conceptual designs and order-of-magnitude cost estimates should be prepared for solution mined hazardous waste retention facilities using both dome and bedded salt formations. These studies should include an evaluation of waste form (i.e., solution, slurry, and/or in-situ solidification). These studies should provide a viable hazardous waste disposal scheme for the most difficult to manage wastes; wastes that remain after all economical means have been employed to prevent, reduce, neutralize, or otherwise render them non-hazardous.

ACKNOWLEDGMENTS

The project team wishes to acknowledge Project Officer Mr. Carlton C. Wiles of the Land Pollution Control Division, Hazardous Waste Engineering Research Laboratory, Cincinnati, Ohio, for his support and guidance throughout the study.

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FACTORS AFFECTING STABILIZATION/SOLIDIFICATION OF HAZARDOUS WASTE

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ABSTRACT

The stabilization/solidification of toxic wastes involves a series of chemical and/or physical treatment procedures. The waste is normally treated so as to complex or bind the toxic elements in a stable, insoluble form or to entrap the waste material in a crystalline matrix. A hazardous waste may contain many constituents that could interfere with the binding process. This recently initiated project is concerned with identifying possible interfering mechanisms between particular compounds and waste binding systems.

A synthetic sludge was produced containing parts per million concentrations of cadmium, chromium, mercury and nickel. The sludge is mixed with increasing concentrations of interfering compounds and then with selected waste binders. Various waste mixtures will be evaluated to determine the particular interfering effects on stabilization/solidification processes. This paper provides an interim report on the progress of the study.

INTRODUCTION

The Environmental Protection Agency (EPA) is responsible for evaluating the suitability of hazardous waste for land disposal and for the examination of hazardous waste delisting petitions under the Resource Conservation and Recovery Act (RCRA) regulations. A thorough understanding of the potential behavior of stabilized/solidified waste is necessary to make judgements as to the effectiveness of contaminant containment. There are several methods available for the stabilization/solidification (S/S) of many hazardous wastes. The complexity of some wastes is such that some of the chemical components of the waste may interfere with the proposed S/S process and cause an undesired phenomenon (e.g. flash set, set retardation, spalling, etc.). There is a lack of quantitative data concerning the effects of these interfering components upon the particular S/S process. A study of the factors affecting S/S of hazardous waste is designed to help fill that data void.

Background

Experience from the cement and asphalt industries, as well as radioactive waste solidification, has demonstrated that small amounts of some compounds can seriously reduce the strength and containment characteristics of binder/waste mixes used in S/S technologies. The common binding materials in waste S/S systems are derived from industrial cement and asphalt materials and some data are available on the effects of impurities on strength, durability, and permeability of structural cement mixtures. The cement industries have had to specify the types of cement, aggregate, and accelerators or retarders that will be permitted in portland concrete specifically because additives affect performance. Radioactive waste processors have had to develop limits for particular components that reduce the effectiveness of S/S of their wastes. In some cases these interfering materials can produce set retardation so that hardening does not occur. In other cases, a waste constituent may cause a flash set or flash hardening so that effective mixing of the waste and binder cannot occur. Waste components that chelate or complex toxic constituents may accelerate their leaching even if the waste is successfully solidified. Some waste constituents can react to cause swelling or disintegration of the solidified mass after setting. The effects of these interfering materials are often disproportionate to the amounts present in the waste.

Research that clarifies the relationship between the composition of the waste and performance of the binder can greatly help in evaluating the hazards of the materials. For example, minor organic components in a waste stream can significantly retard the cementing reaction, but these specific materials can be limited in a processor's treatment permit to assure that binding reactions occur, and that waste containment is not compromised. Similarly other deleterious compounds that cause flash sets or expansion reactions can be excluded where reactive binders are proposed.

When waste is placed for final disposal, there is always a possibility of the waste being unearthed by erosion or later excavation. The impact of stabilized/solidified material on its surroundings after exposure can be significant. The effect that interfering substances have on the durability of stabilized/solidified waste is a question of paramount importance.

PURPOSE

The purpose of this study is to develop technical background data on the compatibility of critical waste constituents with various waste binding agents (portland cement, portland cement with additives, lime-pozzolan cement, gypsum cement). Specific supporting objectives are as follows:

(1) Perform physical tests and chemical leaching tests on waste/ binder mixtures.

(2) Investigate the relationship between the amount of interfering substance and the significance of the interference to waste S/S reactions.

(3) Evaluate the durability of specific waste/binder mixtures to long-term outdoor weathering.

(4) Participate in the joint Environment Canada/USEPA/Industry project to investigate test methods for use in evaluating solid wastes.

This paper will report on the results and progress made as of December 1984.

APPROACH

The study is being conducted in two phases (Figure 1). The first phase (Phase I) was completed in December 1984 and included the following tasks:

(1) Conduct a literature search and report on potential factors affecting solidification processing of hazardous industrial and radioactive wastes.

(2) Selection of four waste/binder systems and ten potential interference materials.

(3) Formulation and processing of stock synthetic waste to be used in preparation of test specimens.

(4) Screening for various waste-to-binder ratios.

Phase II of the study will be conducted over a 24-36 month period. Samples of the selected waste/binder system and interfering materials will be tested and evaluated by both the U. S. Army Engineer Waterways Experiment Station (WES) and the Environment Canada. Samples of stabilized/solidified waste will be exchanged and tested for various physical properties and chemical leaching quality.

PROBLEMS ENCOUNTERED

The first experimental difficulty encountered in this study was the requirement to produce a synthetic waste sludge that was both reproducible and had stable physical and chemical characteristics. A hydroxide metal sludge filter cake has been formulated that meets this study requirement. The cake is prepared by taking a solution of metal salts, precipitating the metals with lime, and then vacuum filtering to produce a consistent filter cake of approximately 25 percent solid content.

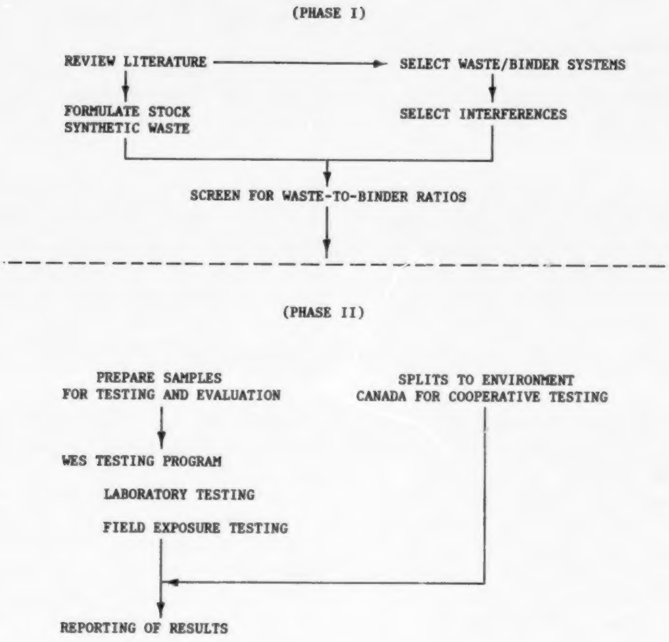
(PHASE I)
REVIEW LITERATURE
SELECT WASTE/BINDER SYSTEMS
FORMULATE STOCK SYNTHETIC WASTE
SELECT INTERFERENCES
SCREEN FOR WASTE-TO-BINDER RATIOS
(PHASE II)
PREPARE SAMPLES FOR TESTING AND EVALUATION
SPLITS TO ENVIRONMENT CANADA FOR COOPERATIVE TESTING
WES TESTING PROGRAM
LABORATORY TESTING
FIELD EXPOSURE TESTING
REPORTING OF RESULTS

Figure 1. Simplified Study Approach.

RESULTS

Literature Survey

A review of the literature on solidification processing of waste showed that published information is limited which specifically relates to organic and inorganic interference phenomena in waste binding processes. Since most waste S/S systems incorporate various cement configurations, some inferences may be made between admixtures in cement chemistry and certain interferences in S/S processes.

Additives in Portland Cement Concrete

In the production of portland cement concrete, the use of chemical additives to control setting times, to reduce water requirements, to entrain air, and to create many other beneficial effects is common practice. The changes in properties affected by additives are assumed to reflect fundamental changes in the hydration products, especially the crystalline calcium silicate hydrate. The chemical and physical development of concrete is complicated. The interfering effects of waste constituents on setting reactions compound the complicated nature of cement chemistry.

The influence of a limited number of organic compounds on the microstructure and engineering properties of hydrated cement has been investigated (Ramachandran, 1971). These compounds include sugars, lignosulfates, carboxylic acids, triethanolamine, and others. One inorganic substance, calcium chloride, an inexpensive and effective accelerator, has been extensively studied (Ramachandran, 1971). Calcium chloride accelerates hydration of tricalcium silicate. Morphological alteration of the hydrated calcium silicate gel and calcium hydroxide, along with changes in CaO/SiO_2 and H_2O/SiO_2 ratios, surface area, and pore-size distributions have been documented (Ramachandran, 1971; Odler, 1971; and Collepardi, 1972).

Interfering Mechanisms

In this section attention is given to conceptual models of interfering mechanisms. These models include interference via adsorption, complexation, precipitation, and nucleation.

Adsorption

One possible interfering mechanism is adsorption of additive molecules by crystalline particles thereby blocking the normal hydration reactions. Studies (Young, 1970) have shown that the retarding effect of organic compounds are related to their adsorption on metastable hexagonal calcium aluminate hydrates. The organic compound inhibits crystal growth and conversion to calcium aluminate hydrates. The inhibiting effect roughly correlates with the number of hydroxyl, carboxylic, and carbonyl groups in the organic molecule. Hansen (1952 and 1959) noted the effect of two particular families of organic compounds, lignosulfonic acid derivatives and hydroxylated carboxylic acids, on setting reactions. Lignosulfonates are strongly adsorbed onto tricalcium aluminate (Blank, et al., 1963). The adsorption of calcium lignosulfonate onto tricalcium aluminate results in a relatively thick film or layer. The strong adsorption and thick layering of lignosulfonates onto tricalcium aluminate is indicative of a chemical reaction involving the organic and tricalcium aluminate hydration product. Taplin (1962) found retarding effects from aliphatic and aromatic dicarboxylic acids (e.g. maleic acid). In alkaline solutions where maleic acid has no hydroxyl group for a hydrogen adsorption bond, chelation may be the interference mechanism.

Although adsorption of organic retarders is primarily on tricalcium aluminate, retardation is due to adsorption on tricalcium silicate. There is no evidence of adsorption onto anhydrous surfaces. Organic additives can have an important bearing on reaction rates during cement hydration.

Complexation

Taplin (1962) related the retarding activity of organic compounds to the proximity of oxygen atoms to carbon atoms. He observed that the compounds with oxy-functional groups, in close proximity to each other, were also indicative of retarder effectiveness, and that chelation to metal ions could be an important factor in set retardation. Calcium ions can chelate with various hydroxyl or carboxylic acids, but the retarder or accelerators (respectively) are so dilute that the complexation of the calcium is not an important factor (Young, 1972).

The effects of complexing calcium are more significant when the additive to affected-ion ratio is very large and when the affected ion is important to the setting system. Such would be the case for the aluminate and ferrite ions. Researchers (Kalousek, 1943, and Roberts, 1967) have shown that the addition of sucrose increases the concentration of alumina and calcium ions to above normal levels. Experiments (Young, 1972) with tricalcium aluminate indicate that 1 percent by weight additions of sucrose, succinic acid and tartaric acid increase the amounts of calcium and alumina in solution at first, but concentrations later decrease to normal or below normal. Silica concentrations are also increased when additives that affect alumina concentrations are used. Apparently, conditions in a cement paste are favorable to aluminate, ferrite and silicate ion complexation. It is possible that complexation delays the formation of hydration products. When cement forming ions are kept in solution by complexation, hydration barriers are established that retard the set.

Precipitation

The formation of insoluble precipitates by additives reacting with cement compounds is conceptually not a realistic mechanism of admixture interference. Certainly the formation of insoluble compounds could impede water transport, solubility, and subsequent hydration reactions. However, if retardation is due to precipitation, then the process should be non-selective. In this case, tricalcium silicate and aluminate would both release calcium ions and the resulting effect on setting should be equally weighed among both compounds.

Nucleation

The inhibition of nucleation of crystalline calcium hydroxide by soluble silica is believed to be the self-retarding set feature of tricalcium silicate hydration. Growth of a crystalline matrix is retarded by the adsorbed silica ions when a hydrated calcium silicate gel layer results in a diffusion barrier to calcium hydroxide. Eventually crystal growth results in the adsorbed silica being trapped in the crystalline matrix as the hydration process continues. Prismatic growth of calcium hydroxide results from differential adsorption of silica on calcium hydroxide crystal faces (Young, 1972).

It is postulated that organic retarders act much the same as silica ions being adsorbed onto the calcium hydroxide nuclei. However because of more retarders being solubilized, the organic retarders are much more effective in being adsorbed and more completely cover crystal growth surfaces. The resulting retardation of crystal growth causes the formation of more crystallite nuclei in the saturated solution. The net effect of crystal growth on this many nuclei is acceleration of tricalcium silicate hydration following the initial retardation period (Young, 1972). A one percent (by weight) of a strong retarding agent, completely inhibits tricalcium silicate hydration. Addition of prehydrated tricalcium silicate has been found to overcome the effect of the organic retarder indicating the merit of an adsorption/nucleation model.

Effects of Organic Compounds on Physical Properties

Accelerators

Any compound that promotes the nucleation and growth of calcium hydroxide will be an effective accelerator. The most commonly used organic accelerator of hydraulic cement is triethanolamine (TEA). At addition rates less than 0.06 percent by weight of cement, TEA is an accelerator. At greater dosages, TEA is a retarder. Calcium formate is another common accelerator. The addition of calcium chloride to the cement mixture accelerates crystal growth by eliminating the necessity for ions to move from the Tricalcium silicate structure into solution. Both TEA and calcium formate have been shown to improve the 28 day compressive strength of Portland cement concrete. Formaldehyde and paraformaldehyde decrease the 28 day compressive strength at higher dosage rates (Rosskopf, et al., 1975).

Retarders

Early strengths are lower in concrete specimens with retarders than those without. However, as the age of the specimens increase, those containing retarders generally have higher compressive strength and comparable flexural strength. The drying-shrinkages of specimens containing retarders are also comparable to those without retarders. It is interesting to note that soluble calcium salts which provide anions that adsorb onto the calcium hydroxide crystal surfaces cause a retarding effect (e.g. calcium nitrate).

Waste Binding Systems and Interferences Selected for Study

Portland cement, lime-pozzolan, and gypsum cement have been used in conjunction with other additives (e.g. flyash) to stabilize/solidify industrial waste sludge. Lime-pozzolan mixtures are frequently used to convert liquids and semi-solids to a solid form. Systems selected for testing are those found in use in waste treatment and judged to have potential widespread application. Type I Portland cement and Type I Portland cement with flyash were chosen for immediate investigation. A lime-pozzolanic cement and a gypsum cement formulation will be used in subsequent experimentation.

Ten (10) materials with properties known or suspected to interfere with the S/S process were chosen for study. The frequency and concentration with which these materials are commonly found in waste streams considered for S/S processing were also considered. The interfering materials selected for study are: (1) oil and grease; (2) light weight oil; (3) phenol; (4) sulfates; (5) strong base; (6) pesticide; (7) degreaser; (8) lead; (9) copper; and (10) zinc.

Synthetic Waste Sludge Development

The synthetic sludge to be used in this study was generated by treating a synthetic wastewater containing cadmium, chromium, mercury, and nickel with hydrated lime to yield a hydroxide sludge containing these metals in concentrations at least 100 times that necessary to cause rejection on the basis of the EP Toxicity Test (eg. 100 ppm Cd, 500 ppm Cr and Ni, 20 ppm Hg). Synthetic wastewater was prepared in 500-gallon batches using the nitrate salts of the above metals. After allowing sufficient mixing time for equilibration, hydrated lime was added in a sufficient quantity to produce a pH of at least 10. Following a period of rapid mixing and slow mixing to allow for floc formation, the mixture was maintained under quiescent conditions for a minimum

TABLE 1. REQUIRED SAMPLES FOR STABILIZATION/SOLIDIFICATION PROJECT-FY85

Processes: Portland cement and portland cement/flyash
Interfering Agents: Oil and Grease, oil, phenol, sulfate, caustic, pesticide, degreaser, lead, copper, and zinc

	Control (0 Percent)				2 Percent				5 Percent				8 Percent					
	Test Period				Test Period				Test Period				Test Period					
Test	24 hr	7 day	28 day	Specimens	24 hr	7 day	28 day	Specimens	24 hr	7 day	28 day	Specimens	24 hr	7 day	28 day	Specimens	Total Specimens	Required Samples
EP (dupl)			X	2			X	2			X	2			X	2	8	0*
Cone Index (dupl)	X	X	X	6 (2 x 3)	X	X	X	6	X	X	X	6	X	X	X	6	24	24
Unconfined compressive strength (trip)	X	X	X	9 (3 x 3)	X	X	X	9	X	X	X	9	X	X	X	9	36	36
Weight/ density (dupl)			X	2			X	2			X	2			X	2	8	0*
Wet/Dry (15 cycles) (dupl except for control)			X	1			X	2			X	2			X	2	7	7 (4 in. diam)
X-ray and SEM (single except for control, test every third control)			X	0 or 1			X	1			X	1			X	1	3 or 4	0*
Permeability (single)			X	1			X	1			X	1			X	1	4	4 (2.8 in. diam)

* The weight/density measurement will be obtained using the 28-day Cone Index (CI) samples. After the CI is obtained, the samples will be broken into pieces and subsamples will be taken for the EP and X-ray/SEM tests.

period of 24 hrs to provide for settling and sludge accumulation. Clear water above the sludge was decanted and the sludge transferred to a holding tank in preparation for dewatering. Dewatering to approximately 25% solids was achieved using a drum vacuum filter. Approximately 150 pounds of sludge is produced in each batch. Fresh sludge will be prepared on a regular basis so that all sludge used in the interference portion of the study will have a maximum age of seven days.

Work to be Conducted in FY 85

The initial laboratory work to be conducted in FY 85 will involve development of the required amounts for the S/S additives used in the portland cement and portland cement/flyash processes. These amounts will be established by preparing serial batches of solidified sludge, allowing representative samples to cure for 28 days, and determining the unconfined compressive strength (UCS) of each sample. A baseline dosage rate will be determined. The baseline dosage is the minimum dosage required to produce a UCS of 100 psi using a minimum amount of additives. Work will also be conducted on the installation and calibration of equipment to be used later in the study, documentation of sample preparation and testing procedures, and the development and verification techniques for preparing and adding interfering agents to the synthetic waste sludge.

Work on preparation of solidified specimens of sludge containing the interfering agents is scheduled to begin in January 1985. Work will be conducted on one S/S process at a time. A list of the samples to be prepared and corresponding testing to be conducted is presented in Table 1. As indicated, aliquots of four sludge/interfering agent mixtures (0, 2, 5, and 8 percent interfering agent by weight) will be solidified and test samples prepared. All four aliquots will be obtained from the same batch of sludge. Testing will include EP, cone index, UCS, weight/density, wet/dry durability, X-ray and scanning electron microscopy, and permeability. Cone index and UCS testing will be conducted on samples aged for 24 hrs, 7, and 28 days. Other testing will be conducted on 28-day samples. Sample preparation for each S/S process will require approximately 20 weeks. Data evaluation will be conducted on a continuing basis as test results become available.

ACKNOWLEDGEMENTS

This study is part of a major research program that is now being conducted by the U. S. Army Engineer Waterways Experiment Station and funded by the Environmental Protection Agency, Municipal Environmental Research Laboratory, Solid and Hazardous Waste Research Division, Cincinnati, Ohio, under Interagency Agreement DW96930146-01. Carlton C. Wiles is the EPA Project Officer for this research area.

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CONSTRUCTION QUALITY ASSURANCE FOR HAZARDOUS WASTE DISPOSAL FACILITIES

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ABSTRACT

This paper describes the elements of a Construction Quality Assurance (CQA) Plan for various components of hazardous waste land disposal facilities. These components include synthetic membranes, low-permeability soils, leachate collection systems, layered covers, and dikes. The paper refers to research activities in the development of the CQA program discussed in the Environmental Protection Agency's (EPA) draft technical guidance on double liner systems. The CQA elements covered include: (1) responsibility and authority, (2) personnel qualifications, (3) observation and tests, (4) design of a sampling plan, and (5) documentation.

INTRODUCTION

On November 8, 1984, the Hazardous and Solid Waste Amendments of 1984 (HSWA) were enacted. HSWA contains minimum technology requirements for land disposal facilities, including requirements for liner and leachate collection systems. For certain landfills and surface impoundments, HSWA requires two or more liners and a leachate collection system above (in the case of a landfill) and between such liners. This paper refers to such liner and leachate collection systems as "double liner systems."

On December 20, 1984, EPA issued draft technical guidance regarding the HSWA minimum technology requirements for liners. The purpose of EPA's draft technical guidance is to present design, construction, and operation specifications that can be used to comply with HSWA. The guidance is organized in sections and discusses:

- Primary leachate collection and removal systems;
- Double liner specifications;
- Secondary leachate collection systems between the liners; and
- Construction quality assurance.

There is little debate about the need for a construction quality assurance program for double liners and other components of hazardous waste land disposal facilities. The components, in the case of double liners, must be designed, constructed, and operated to prevent flow through the liners and maximize removal of any leachate, thereby protecting human health and the environment.

Results of recent EPA studies concluded that the most critical problems associated with liners at existing facilities result from the lack of adequate quality control during the installation. For example, the major causes of synthetic liner failures were related to inadequate seams and material breaches (e.g., tears) rather than from waste/liner incompatibility. This indicates the need for both better construction practices and better CQA. One of these studies, the "Liner/Locational Analysis Project," (EPA, 1984) concluded that available techniques can be used to install synthetic and clay liner systems that meet the design specifications. However, a comprehensive inspection and audit program is needed to ensure attainment of design specifications. Proper installation of either a synthetic or clay liner within a hazardous waste land disposal facility requires strict adherence to precise installation procedures to achieve facility performance specifications.

BACKGROUND

EPA initiated its current construction quality assurance program in late 1983. The program is divided into two parts (1) regulations/guidelines that specify the wide use of CQA during construction and outline EPA's CQA program (as found in the draft technical guidance) and (2) guidance that addresses the elements of a site-specific CQA plan prepared by an owner/operator. Part 1 is being developed by the Office of Solid Waste. The site-specific guidance of Part 2 is being developed by the Hazardous Waste and Engineering Research Laboratory (HWERL). HWERL is using a two-phased research approach to meet the goals on the site- specific CQA guidance. Phase One of the research program is to provide currently available information on CQA for each element of the CQA program. Phase Two is to develop comprehensive construction quality assurance guidance through a research program to gather information on areas not addressed in Phase One.

HWERL's Phase One activities started in the spring of 1984. Research contractors prepared documents on five separate components of a hazardous waste land disposal facility: low-permeability soils, synthetic membranes, dikes, leachate collection systems, and cover systems. Each contractor assembled this information into a preliminary draft document. These preliminary draft documents were then reviewed by EPA, the contractors, and consultants. Comments from these reviewers were incorporated into revised drafts. The five separate reports are being merged into document discussing all five facility components. This draft document is planned to be available for public comment in June 1985. This paper briefly summarizes the information that is included in the Phase One site-specific CQA guidance.

ELEMENTS OF A CONSTRUCTION QUALITY ASSURANCE PLAN

KEY TERMS

Construction quality management, as discussed herein, is a process that uses scientific and engineering principles and practices to assure, with a reasonable degree of certainty, that a constructed hazardous waste land disposal facility meets design specifications. Construction quality management should begin during design of the facility and continue through the completion of facility construction.

Managing construction quality is the responsibility of the facility owner/operator and his supporting organizations. It consists of two components, defined as follows (adapted from ASTM):

o Construction Quality Assurance (CQA)--A planned program of overview activities, the purpose of which is to provide assurance that construction quality control is being effectively implemented. The system involves a continuing evaluation of the adequacy and effectiveness of the construction quality control plan, the activities carried out, the data collected, and the interpretation of the data made in response to the construction quality control plan. For hazardous waste land disposal facilities, this program may involve verifications, audits, and evaluations of the factors that affect the installation, inspection, and performance of a facility in order to assure that the facility meets the design specifications.

- Construction Quality Control (CQC)--A series of planned and unplanned inspection activities, the purpose of which is to provide the level of construction quality that will result in a facility that meets the design specifications. The overall system involves integrating several related factors including: the proper selection of specified materials, installation to meet the full intent of the specification, and inspection to determine whether the resulting product is in accordance with the specification.

The Construction Quality Assurance plan, as discussed in this paper, is the written approach followed by the owner/operator and his supporting organizations in attaining and maintaining high quality in the construction of a hazardous waste land disposal facility. The CQA plan documents the owner/operator's commitment to CQA and should be tailored to the site-specific conditions at the facility to be constructed. While the overall content of the CQA plan will depend on the site-specific nature of the facility, at a minimum the plan should include the following elements:

(1) areas of responsibility and lines of authority in executing the CQA plan;

(2) qualifications of CQA/CQC personnel;

(3) types of observations and tests to be performed under the CQA plan;

(4) design of a sampling plan, including sampling frequency, acceptance/rejection criteria, and corrective action procedures; and

(5) documentation procedures and recordkeeping.

(1) Responsibility and Authority

The principal parties involved in permitting, designing, and constructing a hazardous waste land disposal facility will include the permitting agency, the owner/operator, the design engineer, CQA/CQC personnel, and one or more construction contractors. Except for the permitting agency, the principal parties will not necessarily be independent of each other (e.g., the owner/operator may also be the construction contractor; the design engineer may also be the construction contractor; the CQA/CQC personnel may be employees of the owner/operator or employees of the design engineer) Regardless of the relationships among the parties, it is essential that the areas of responsibility and lines of authority for each party be clearly established as the first element of the CQA plan.

As an example, the responsibility of the CQA/CQC personnel is to execute the CQA program. In order to do this, the CQA/CQC personnel should include a CQA officer and one or more CQC inspectors. The specific responsibility and authority of each of these persons should be clearly defined in the CQA plan and associated contractual arrangements with the facility owner/operator. For the CQA officer, specific responsibilities may include:

- serving as the owner/operator's or design engineer's liaison with construction contractor personnel and helping to interpret and clarify contract documents;

- educating construction and inspection personnel on job requirements;

- reviewing design drawings and specifications for clarity and completeness;

- scheduling site inspections and testing;

- directing, overseeing, or checking the CQC inspector when performing site inspections and tests;
- providing CQA reports to the owner/operator on the results of inspections and tests including:
    - reviewing observation records and test results;
    - advising the owner/operator or design engineer of work that the CQA officer believes should be corrected, rejected, or uncovered for observation, or that may require special testing, inspection, or approval;

      rejecting defective work and specifying corrective measures when authorized by the owner or design engineer.

For the CQC inspector, specific responsibilities may include:

- conducting onsite observations of the work in progress to assess compliance by the contractor with the plans, specifications, and construction-related contractual provisions for the project;
- conducting field and laboratory tests;
- reporting, to the CQA officer, the results of all inspections including work which is not within contractual quality or fails to meet contract requirements;
- monitoring the reviews and tests conducted by the contractor as required by the specifications and contract; and
- verifying that tests, equipment, and system startups are conducted by qualified personnel and proceed according to standardized procedures defined by contract documents.

(2) Personnel Qualifications

The effectiveness and ultimate success of the hazardous waste land disposal facility will greatly depend on the qualifications and performance of the CQA/CQC personnel. The CQA officer and the CQC inspector should possess the training and experience necessary to fulfill their responsibilities. The CQA officer should possess a bachelor's or advanced degree in engineering, professional registration, and sufficient practical experience in land disposal facility construction and construction site experience to demonstrate expertise in successfully implementing CQA-related activities. The CQC inspector may have an associate's or bachelor's degree in engineering and should have sufficient years of practical experience in construction inspection to have established a record of successful inspection-related activities. The CQC inspector should be familiar with specific practices in the field relating to construction techniques, codes and regulations regarding material and equipment installation, site safety, and testing.

(3) Observations and Tests

The observations and tests that will be conducted during hazardous waste land disposal facility construction should be described in the CQA plan. Inspection of materials and workmanship during all phases of construction is aimed at determining whether the properties, composition, and/or performance of the material or installed components are within limits specified in the design. The following subsections address the CQA/CQC activities should take place during preconstruction, construction, and postconstruction periods for a hazardous waste land disposal facility.

Preconstruction--

The first activity under preconstruction CQA/CQC is to assess the adequacy and clarity of construction design drawings. The CQA officer should review the design to ensure that CQA can be performed unambiguously. The construction design drawings and specifications need to be clear and understandable from the standpoint of both the onsite CQC inspectors and the contractor. If the design is deemed misleading or unclear by the CQA officer, it should be returned to the design engineer for clarification and/or modification.

Prior to construction, the CQA officer must also assess the capabilities of the construction contractor's personnel so that he can determine the amount of supervision that will be needed during construction operations. The contractor's prior performance in general earthwork activities, his experience in construction of hazardous waste land disposal facilities, and his experience in working with the specific soil, synthetic membrane, and equipment to be used for constructing the facility components in question need to be assessed. A preconstruction training plan for field inspection personnel also should be included in the CQA plan.

The CQA plan must provide assurance that all materials used to construct the facility meet the design specifications. This includes inspecting material type and quality and factory-installed seams of synthetic membranes, determining the index properties of soils, and evaluating the quality and assembly of equipment such as pumps, pipes, and concrete structures. Provisions must also be made for the storage of all construction materials, especially those that may be altered by outside conditions such as rainfall, extreme temperatures, and sunlight.

Another preconstruction activity that should be specified in the CQA plan is assuring the suitability of the equipment and methodology to be used to install the facility components. This may include testing of the seaming or bonding equipment and procedures to be used in synthetic membrane installations and the methods to be used for the joining of drainage pipes. EPA's December 20, 1984 draft technical guidance on double liner systems strongly suggests that, when compacted earth fill is to be installed, as in a low-permeability soil liner, dike, or cover system subcomponent, a representative test fill of soil be compacted to see if the specified density/density/moisture content/permeability relationships, determined in the laboratory, can be achieved in the field with the compaction equipment to be used. Test fill dimensions must be sufficient to accommodate the compaction equipment. The draft technical guidance also suggests that two or more lifts be compacted in the test fill to check the methodology to be used to tie together lifts. The test fill also is used to determine the number of equipment passes (or amount of compactive effort) needed to achieve the specified permeability and to evaluate the ability of mixing equipment to break up large clods of soil. Field permeability tests should be conducted on the test fill to assure that the compactive effort applied in the field will result in the same (or lower) permeability that was observed during laboratory soil testing.

Construction--

Testing of materials during construction is aimed at determining whether the properties, composition, and/or performance of the materials or installed components are within limits specified in the design. Testing of equipment may include evaluating the quality and assembly of materials and performance of the equipment under specified test conditions.

An important activity specified in a CQA plan for a hazardous waste land disposal facility is visual observation of the construction process, including personnel performance, by the quality control inspector. Visual observations by an experienced inspector, coupled with a soundly developed and implemented testing plan, will assure that the facility components are installed as specified, that any potential problems are identified in a timely manner, and that proper corrective actions are implemented.

Postconstruction--

The CQA plan should specify that after completing the various facility components, CQA/CQC personnel should conduct a final inspection, including completion tests, to make certain that each component was installed according to design specifications. Examples of items to be inspected include sidewall and bottom slopes; liner coverage, thickness, and permeability; primary and secondary collection layer, and protective cover layer thicknesses and coverage; placement and integrity of pipes and concrete structures; seaming and anchoring of synthetic membranes; and operation of all pumps and electrical equipment.

Field permeability measurements may be conducted on the completed low-permeability soil barriers as a final CQA check by setting up several field permeameters over the specified area. To check for leaks in installed synthetic membrane liners and low-permeability soil liners, the facility can be filled or partially filled with water, and the volume lost from the site can be measured, after accounting for evaporation, or by measuring the volume of liquid in the collection system between the liners. Flooding with water, when feasible, may be the best way to assure that the entire site will function according to specifications after it is filled, assuming that no waste/liner compatibility problems occur.

After facility closure and the installation of the cover system is complete, similar completion tests must be made. For a cover system, items to be inspected include such things as verification of thicknesses and slopes of cover layers (e.g., low-permeability soil barrier, drainage layer, filter layers, soil cover), seals around penetrations such as gas vents, tying together of the cover and liner systems, drainage system operation, and growth of vegetation for erosion control.

(4) Sampling Program Design

The fourth element of a CQA plan is the design of the sampling program. Parts of a sampling program include (Willenbrock, 1976; Miller-Warden Associates, 1965):

o The quality characteristic(s) to be evaluated;

o The block size: a defined quantity of material or of construction specified for acceptance or rejection;

o Number of sample units or measurements per block;

o Location(s) of sample units or measurements;

o Sampling and testing procedures;

o Method of calculation and analysis of data; treatment of outliers;

o Acceptance/rejection criteria; and

o Corrective measures to be taken in the event of noncompliance.

The evaluation of the quality of materials and workmanship during the construction of a hazardous waste land disposal facility relies on an inspector's ability to make judgments based on the results of appropriate tests and observations. Two types of inspection can be used during hazardous waste land disposal facility construction: 100-percent inspection and sampling inspection. It is important to remember that in the case of liner systems at hazardous waste land disposal facilities the consequence of an oversight of just one major tear or unwelded seam can be significant. It is especially necessary to utilize a sampling program that yields a high degree of certainty that all seams are properly completed.

Visual inspections and simple nondestructive tests of workmanship, materials, and system performance are two types of 100-percent inspection activities. Examples of 100-percent inspection include visual inspection of materials (e.g., soils, synthetic membranes, geotextiles) on arrival at the facility and during installation; observation of drainage layer, barrier layer, or cover layer thickness and coverage; visual inspection of synthetic membrane seaming and anchoring; and visual inspection of the prepared foundation, or any other completed facility component. Nondestructive tests include: testing of synthetic membrane seaming and anchoring, collection system pipe joints, pump function, electrical connections, and filling a containment facility as a final leak test of the entire structure.

Testing of earthwork quality (for low-permeability soil barriers, dikes, drainage layers, and soil covers) to meet performance criteria requires sampling inspection in which a sample is removed from, or isolated within, the structural component and subjected to destructive testing in the field or laboratory. Current earthwork construction quality control practice relies on the inspector's observations and judgment for acceptance. Sampling inspection is always required, however, to confirm the inspector's judgment and to determine compliance with performance criteria.

A sampling plan must be designed with consideration of the variability that is acceptable in the workmanship or material. Since the objective of CQA is to assure confidence in the long-term performance of the system, the range of acceptable variability may be narrow and acceptance-rejection criteria may be very restrictive. Acceptable variability must be based upon well-defined engineering principles and detailed knowledge of the effects of altering the variables. This is particularly important in earthwork construction where variability in the properties of the soil source is the rule rather than the exception. That variability can profoundly affect the final soil liner properties. The source soil will require sampling and testing, followed by revision of the density-moisture-compactive effort criteria in order to consistently maintain an acceptably low permeability of the liner as constructed. The frequency of testing and acceptance-rejection criteria revisions should be inversely related to the uniformity of the materials being tested, and may change during the construction period, as conditions warrant.

Statistical sampling plans are not widely used at present in CQA/CQC for hazardous waste land disposal facilities. However, there are benefits to their use because statistical methods recognize and consider the variability that is an established fact for all construction operations. The use of statistical sampling plans may result in more confidence and consistency than currently exist in quality assurance programs for hazardous waste land disposal facility construction.

Statistical methods may be used to help define sample frequency and locations and to analyze testing data and its meaning with respect to established acceptance-rejection criteria. Statistical sampling, based on expected variability, requires some estimate of that variability. Engineering experience is critical, particularly in the geotechnical area, where substantial variability may be expected. A CQA officer or CQC inspector with such experience is a very important attribute of a CQC plan.

Statistical sampling methods for earthwork quality control (geostatistics) have been developed by the U.S. Bureau of Reclamation and the U.S. Army Corps of Engineers (USBR, 1974; U.S. Department of the Army, 1977; Davis, 1966; Turnbull et al., 1966; Willenbrock, 1976). These documents include statistical methodology to establish a degree of confidence for a testing program. Other discussions of statistical methods for earthwork quality control may be found in Winterkorn and Fang (1975), Selig (1982), Wahls et al. (1968), Jorgenson (1971), and Kotzias and Stamatopoulos (1975).

In the event of noncompliance, when material or work is rejected because observations or tests indicate that it does not meet the specified acceptance criteria, corrective measures must be implemented. For material subject to 100-percent inspection, substandard material is simply rejected. When workmanship subject to 100-percent testing is rejected (e.g., synthetic membrane seams), it must be redone until it meets specifications. For workmanship in question because of an inspector's observations, additional testing of the component may be necessary for verification prior to rejecting the block of work and specifying corrective measures. When failures are found during sampling inspections, those failures need to be repaired and additional sampling generally is required.

(5) Documentation

An effective CQA program depends to a large extent on recognizing all of the construction activities that should be inspected and the assignment of responsibilities to personnel for the inspection of each activity. This is most effectively accomplished by documenting CQA/CQC activities. Through documentation, the CQC inspector will be reminded of the factors to be monitored, inspected, or tested, and will note through required descriptive remarks, data sheets, and checklists signed by him that the inspection activity has been accomplished.

Standard reporting procedures should include daily preparation of field data sheets. These data sheets should include such information as the date, weather conditions, construction operations in progress, the location and results of CQA/CQC tests conducted, description of any problems encountered, corrective actions, and the signatures of responsible CQA/CQC personnel. A daily summary report should be used to provide the chronological framework for identifying all other reports.

Large projects may benefit by dividing the work into blocks of smaller size and treating each block as a separate inspection area. After completing each block, a specific block evaluation report documenting all construction and corrective action activities may be prepared. These block evaluation reports may then be used to summarize all of the site construction activities. Block evaluation reports as well as all data sheets and daily summary reports should be submitted to the design and construction engineer for evaluation and analysis. Timely submittal of these documents will allow for errors, inconsistencies, and other problems to be detected and corrected as they occur (i.e., when corrective measures are easiest).

Upon completing the project, the facility owner/operator should submit a final documentation report to the permitting agency and retain a copy in the facility operating record. This report should include summaries of all construction activities, observation and test data sheets, problem reporting and corrective measures data sheets, block evaluation reports, deviations from design and material specifications and as-built drawings. This document should be prepared by the CQA officer and included as part of the CQA plan documentation.

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FIELD VERIFICATION OF LANDFILL COVER SYSTEM CONSTRUCTION TO PROVIDE HYDROLOGY ISOLATION

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ABSTRACT

The purpose of this research is to evaluate the effectiveness of multilayer soil cover systems in providing hydrologic isolation of underlying landfill material. Each cover system was comprised of vegetated topsoil, underlain by a coarse-textured soil drainage layer, underlain by a compacted clay soil barrier layer, underlain by several feet of sand to represent landfill material. Three plot areas, each of different soil thickness and moisture sensor placement configuration, were constructed during the early part of this study. Cover effectiveness is to be demonstrated through a comprehensive water balance analysis incorporating standard climatological parameters, spacial and temporal soil moisture movement throughout the multiple layers, and measured flows from surface, wicking, and leachate drains. The data are to be further used to test the ability of the HELP (Hydrological Evaluation of Landfill Performance - EPA) Model to predict the landfill cover performance.

Instrumentation and monitoring are discussed with respect to placement of instruments and frequency of data collection. Monitoring equipment was placed during construction to minimize the placement effects on the integrity of the barrier layer. Soil moisture monitoring sensors were placed to assure the availability of data in both the vertical and horizontal directions and among soil layers. Data collection frequency was selected for the various monitoring devices such that all significant moisture changes were recorded.

Data obtained from the three multi-layer soil landfill covers are described in detail. Design and construction details are presented as necessary to describe the nature of the data. Emphasis is placed on a multi-sensor integrated systems approach to hydrologic monitoring using computerized acquisition, analysis, and evaluation. The integration of monitoring sensors has provided data which describe the functioning of the multi-layered cover systems. Since data interpretation is tempered by instrument sensitivity to changes in soil moisture and temperature, extensive calibration and instrument response testing were accomplished to ensure that sensor variability would be taken into account throughout data analysis. Resistance block calibrations, soil moisture characteristic curves, and soil layer moisture migration rates are presented with procedures, data, variability, and conclusions.

The HELP model is designed to assist in the prediction of multilayered landfill cover hydrologic performance when a specific set of design parameters, climatological inputs, and soil characteristics are provided. Model parameters are identified and assigned recommended values in the users' manual listing. This presentation will include data showing observed versus predicted (by models) values of soil layer moisture content for both drying and precipitation event conditions. Experimental data from this study was applied to refine input parameters of the HELP model for single storm events; observed and predicted values for multiple storm events were also compared.

At the time of preparation of this abstract, evaluation of the capabilities of the HELP model with respect to field data collected from this study was only preliminary. The data suggest that the constructed cover systems are performing as expected, based on data collected over only six months. Further data collection and reduction will allow more definitive evaluation of the hydrologic models. Hydrologic response of the three multi-layer cover system plots will be stressed using the University of Kentucky's rainfall simulator to create intense rainfall events during the future duration of this study.

MAINTENANCE FREE VEGETATIVE SYSTEMS FOR LANDFILL COVERS

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ABSTRACT

The overall objective of this EPA funded project is to investigate water balance relationships in hazardous landfill cover treatments, with the aim of identifying soil/vegetation systems that have promise for long-term stability. We are evaluating the utility of using the USDA CREAMS model as a hydrology modeling tool to assess the performance of site specific vegetation treatments and soil properties.

A successful cover system must be designed to minimize long-term migration of liquids from the waste, minimize erosion, accommodate subsidence, retain vegetation cover integrity, and be relatively maintenance free. These constraints require a cover design that eliminates deep percolation, controls erosion, and utilizes native plant species that are preadapted to local climate and are successful competitors.

The fraction of precipitation input that results in percolation below the root zone is substantially affected by the evapotranspiration component of the water balance in the cover. Thus, our focus has been on quantifying and evaluating those attributes of native plant species that directly affect evapotranspiration: leaf area index (LAI), rooting depth and distribution, and phenology.

Simulations of cover treatments in a humid climate have indicated that even the native plant species offering the greatest potential for removal of normal precipitation inputs over the entire year are not successful in fully controlling percolation. In semi-arid ecosystems, the situation is more complex. Percolation is apparently effectively eliminated in normal precipitation cycles and amounts. But timing and quantities can be such that deep percolation will not be precluded by native plants.

A second phase of the research is currently being implemented. It involves a field scale experimental test of semi-arid ecosystem trench cap cover treatments. The test facility is a closed shallow land burial site at Los Alamos that has several experimental multilayer soil cover treatments on it. These soil profiles have been planted with two warm season native species (a shrub and a grass) at two different densities. Those should produce quantitatively different plant canopies, rooting profiles, and water use strategies. The experimental design and data collection methods will be presented.

AN ELECTRICAL TECHNIQUE FOR DETECTING LEAKS IN MEMBRANE LINERS

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ABSTRACT

An electrical technique for detecting leaks in membrane liners has been developed and tested for the U.S. Environmental Protection Agency. The technique detects and locates areas of the liner where physical integrity has been breached. The technique takes advantage of the high electrical insulating properties of the liner compared with the waste contained above the liner and the soil under the liner. If the liner is physically punctured or separated, the electrical conductivity of the fluid and underlying soil from a current path by which the leak may be detected and located. The technique can be applied on the surface of an existing fluid impoundment and will not damage the liner. It can also be used to survey new membrane liner systems before they are put into service. The technique incorporates digital data acquisition equipment, which eliminates human error in manual reading and recording of measured data, and it is portable to facilitate onsite application by two or three field personnel.

Full-scale testing of the technique was performed at a 0.4 ha (1-acre) fluid impoundment lined with a high density polyethylene membrane. Tests were conducted to detect and locate single and multiple leaks of different sizes and locations. The results, which have been reported by Shultz, et. al.(1), indicate that the technique was successfully used to detect and locate single and multiple artificial leaks as small as 0.03 m (1-inch) in diameter with an accuracy of 1.5 m (5-feet) or less.

A new project was initiated in October 1984 with the following objectives:

(1) Demonstrate the application of the electrical survey technique for detecting and locating leaks in liner systems prior to facility startup;

(2) Demonstrate the application of the electrical survey technique for detecting and locating leaks in liner systems currently in service;

(3) Develop test data for application of the technique to double-lined impoundments;

(4) Determine the relationship between liner electrical resistivity and exposure to selected waste fluids;

(5) Develop a computer model to relate facility design parameters to survey results; and

(6) Assess the field operational procedures used to conduct the surveys on the Institute grounds.

The work presented here consists of: (1) the results to date of field applications of the technique; (2) the results to date of the double liner model studies; and (3) the results of one-sided exposure tests to measure changes in volume resistivity of selected membrane materials.

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(1) Shultz, D.W., B.M. Duff, W.R. Peters, Technique to Assess the Integrity of Geomembrane Liners, U.S. Environmental Protection Agency, Cincinnati, Ohio, 1984. NTIS No. PB 85-122414.

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